

MODEL FOR THE TIME RATE OF LOCAL SEDIMENT SCOUR AT A
CYLINDRICAL STRUCTURE

By

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This document is dedicated to my Mom and Dad;
to my fiancée, Amy; and to Kelli and Timmy.

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KEY TO SYMBOLS

C	transport function coefficient (volume/time/length)
D	pier/cylinder diameter
d_{50}	median sediment diameter
D/d_{50}	normalized sediment size
d_s	instantaneous scour depth
d_{sc}	equilibrium scour depth
n	erosion zone width divided by the cylinder diameter
N_s	sediment number ($N_s = V / \sqrt{(sg - 1)gd_{50}}$)
p	sediment porosity (V_{water}/V_{total})
Q	rate of transport of material by volume (volume/time)
q	rate of transport of material by volume per unit width (volume/time/length)
sg	sediment specific gravity (ρ_s/ρ)
V	depth averaged flow velocity upstream of the structure
V_c	depth averaged critical velocity
V/V_c	flow intensity
V_s	scour hole volume
V_s	volume of sediment solids
V_w	volume of water
y	normalized instantaneous scour depth ($y = d_s/d_{sc}$)

y_0	upstream water depth
y_0/D	flow aspect ratio or flow shallowness
y_b	normalized instantaneous scour depth at which the break point shear stress occurs
y_p	normalized instantaneous scour depth at which the shear stress peak occurs
β_c	half-angle of the perimeter of the erosion zone in radians
ε	minimum flow intensity for which scouring will occur ($\varepsilon = V/V_c _{\min} \sim 0.47$)
ϕ	sediment submerged angle of repose
θ	nondimensional bed shear stress ($\theta = u_*^2/(sg-1)gd_{s0} = \tau/\rho(sg-1)gd_{s0}$)
θ'	effective nondimensional bed shear stress
θ'_0	effective initial nondimensional shear stress at the pile required to initiate scouring ($\theta'_0 = \theta_u/\varepsilon$)
θ_c	critical nondimensional bed shear stress
θ'_p	effective peak nondimensional bed shear stress
θ'_b	effective break point nondimensional bed shear stress
θ_u	nondimensional bed shear stress upstream of the structure
ρ	density of water
ρ_s	density of sediment or bed material
τ	bed shear stress ($\tau = \rho u_*^2$)
τ_0	initial bed shear stress at the structure required to initiate scouring
τ_c	critical bed shear stress
τ_u	bed shear stress upstream of the structure

Abstract of Dissertation Presented to the Graduate School
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May 2003

Chair: D. Max Sheppard

Major Department: Civil and Coastal Engineering Department

We developed a semi-empirical mathematical model for the time rate of local scour at a circular cylinder located in an erodible cohesionless sediment and subjected to an unsteady water flow. The model can be used for both clear water and live bed scour conditions. A knowledge of the structure, flow and sediment conditions and the corresponding equilibrium scour depths is required as input to the model. The scour hole is assumed to have the idealized geometry of an inverted frustum of a right circular cone that maintains a constant shape throughout the scour process. The slope of the sides of the scour hole is uniform and equal to the submerged angle of repose for the sediment. Removal of sediment from the scour hole is limited to a narrow band adjacent to the cylinder where the effective shear stress is greatest. The sediment transport function used in the model is based on commonly used functions for transport on a flat bed.

The effective shear stress in the scour hole, used in the sediment transport equation, is a function of the normalized scour depth (scour depth/equilibrium scour

depth) and the structure, flow and sediment parameters. The function for the shape of the effective shear stress versus normalized scour depth and its dependency on structure, flow and sediment parameters was determined empirically using data from a number of clear water and live bed scour experiments conducted by University of Florida and University of Auckland (New Zealand) researchers. These experiments cover a wide range of structure, flow, and sediments conditions.

Once verified with prototype scale data, the model can be used to estimate such things as the depth of scour that will occur during a time-varying flow event, such as that produced by a hurricane storm surge in coastal waters.

CHAPTER 1 INTRODUCTION

Scour is the removal of sediment (soil and rocks) from the area around or near structures located in moving water. Such scour around pier and pile supported structures and abutments, especially during high flow events (storms, floods, etc.), can result in structural collapse and loss of life and property. Many areas depend on structures subject to scour for access and safe evacuation of these areas during flooding and storms is critical. From a purely economic standpoint, "businesses of all sizes depend on major interstates, city streets and rural roads to move products and services. Therefore, where roads and bridges are temporarily or permanently closed due to damage sustained because of floods, the economy will suffer." (Rhodes & Trent 1993, p. 928)

In 1987, ten people were killed by the collapse of the New York State Thruway bridge across the Schoharie Creek. In 1989, eight people were killed when a section of the U.S. 51 bridge toppled into the Hatchie River (near Covington, Tennessee). Also in 1989, two people were killed when spans of a temporary bridge crossing the Great Miami River near Miamitown, Ohio, collapsed (Gosselin 1997). In 1995, seven people were killed and one injured when the Interstate 5 bridge over Arroyo Pasajero in California collapsed during a flood event. The cause of failure of each of these bridges was identified as scour around the foundation piles.

Brice and Blodgett (1978) studied 383 bridge failures caused by catastrophic floods. Approximately half of these failures were caused by local scour. Although some

of the scour was attributed to the increased local and contraction scour due to accumulation ice or debris, a large portion resulted from erroneous prediction of local scour during design events, specifically flooding events.

Parola et al. (1997) state that "over 400 bridge crossings on federal aid routes and over 2000 non-federal aid bridges were damaged during the 1993 Mid-West flooding." Kamojjala et al. (1995) detailed the damage and costs of bridge failures associated with the catastrophic flood of the Mississippi River in 1993. Two of the 28 failures detailed were directly attributed to local scour (totaling almost \$1,000,000 in damages). Many more of the 2500 bridges requiring Federal Emergency Management Agency (FEMA) assistance also experienced damage due to local scour.

According to the New York State Department of Transportation, from 1980 to 1990, there were eleven New York bridge failures. Five times as much money was lost by local businesses and industry than was actually paid out through highway repair contracts (Rhodes and Trent 1993).

Morris and Pagan-Ortiz (1997) reviewed the bridge scour evaluation programs in the United States. They found that 73 bridges were destroyed by flooding in Pennsylvania, Virginia, and West Virginia in 1985. Seventeen bridges in New York and New England failed in the spring of 1987 due to floods.

Eighty-six percent of the 577,000 bridges in the National Bridge Registry (NBI) are built over waterways. More than 26,000 of these bridges have been found to be scour critical, meaning that the stability of the bridge foundation has been or could be affected by the removal of bed material (Richardson 2002). Over 1000 bridge failures have occurred in the United States over the last 30 years. Over 60% of bridge failures in the

U.S. were due to scour of the bridge foundations (Sumer et al. 2002).

These studies underscore the importance of predicting local scour. Improving the understanding of the local scour phenomena is therefore vital to the engineer responsible for the design of pier foundations.

It is important to be able to accurately predict equilibrium or ultimate local scour. It is likewise important to be able to predict the rate at which local scour occurs. Present Federal Highway Administration (FHWA) methods for predicting local scour use the maximum flow velocity that occurs during a design flow event as input into formulas for calculating equilibrium scour depths.

Controlled experiments have shown that under prototype conditions, the time to reach equilibrium scour depth may be measured in weeks, while these design events may last only days or merely hours (Sheppard 1993). Thus, understanding the temporal dependence of local scour is important in predicting scour under short duration, unsteady design flow events, such as those generated by hurricane storm surges along the Gulf of Mexico and the Atlantic Ocean coasts.

The FHWA has sponsored scour-monitoring programs whose goal is to identify problem areas and take appropriate action before disaster occurs. Once monitoring has identified problem areas, the engineer must implement local scour countermeasures. In this case, knowledge of the variation of local scour with time would help identify which bridges need immediate attention and which simply require continued monitoring.

The objective of this dissertation is to develop a mathematical model of the time dependent processes involved in local scour at a single, simple structure so that insight into the temporal behavior of local scour can be obtained. The mathematical model has

been constructed from basic theory and applied to existing laboratory data from near prototype scale experiments. These data were collected by D. Max Sheppard and his students at the University of Florida, with funding from the Florida Department of Transportation (FDOT) and the Federal Highway Administration (FHWA) (Sheppard 2002b).

Chapter 2 reviews the current understanding of both equilibrium and the temporal behavior of local scour. The various scour mechanisms are defined and described, followed by a review of the literature on the temporal behavior of local scour.

Chapter 3 discusses the experiments conducted by the University of Florida at the USGS Conte Research Laboratory in Turners Falls, MA and at the University of Auckland, New Zealand. The USGS Laboratory experiments include some of the largest pile diameter to sediment diameter ratios ever used under controlled conditions. Analysis of this data is discussed.

Chapter 4 develops a mathematical model for describing the temporal behavior of the local scour from basic principles. Local scour data from the USGS and University of Auckland experiments were used to provide some of the relationships required by the model. Of primary importance is the relationship between normalized "effective" shear stress in the scour hole and the normalized scour depth. The dependence of this relationship on the structure, sediment and flows parameters is discussed in detail.

The model is applied to a number of prototype situations (Chapter 5) in an attempt to shed some light on the problems faced by practicing hydraulic engineers. These numerical experiments also compare the modeled scour to the current practice of calculating equilibrium scour depths using the maximum flow velocity that occurs during

a design flow event. Chapter 6 summarizes the work presented in this dissertation, identifies additional questions, and recommends areas for future study.

Several appendices show details of the development of some of the relationships needed for the model. These relationships include critical shear stress vs. sediment diameter, live bed peak velocity calculations, and shear velocity calculations.

CHAPTER 2 BACKGROUND AND LITERATURE REVIEW

2.1 Mechanics of Local Scour

When a structure is placed in a current, the flow is accelerated around the structure; and the vertical velocity gradient of the flow is transformed into a pressure gradient on the leading edge of the structure. This pressure gradient results in a downward flow that impacts the bed. At the base of the structure, this downflow forms a vortex whose ends are swept around and downstream of the structure by the surrounding flow field. This is the horseshoe vortex; named for its horseshoe shape when viewed from above.

At the water's surface, the flow and structure interaction forms a bow wave known as the surface roller. Beyond the points of flow separation on the structure, wake vortices occur (Figure 2-1).

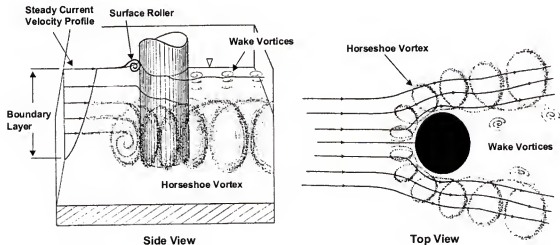


Figure 2-1. Flow patterns associated with local scour

Locally shear stress increases at the bed next to the structure. If the bed is erodible (and the shear stresses are of sufficient magnitude), a scour hole forms around the structure. This phenomenon is known as local or structure-induced sediment scour.

Local scour is classified as either clear water scour or live bed scour. When there is no movement of sediment on the bed away from the structure (i.e., where the influence of the structure on the flow is negligible) the phenomenon is known as clear water scour. Under these conditions, the bed shear stresses away from the structure are less than required for sediment movement (i.e., less than the sediment critical shear stress). At the structure, an initial period of rapid erosion is followed by an equilibrium (reached when the flow alteration caused by the scour hole reduces the magnitude of the shear stresses such that sediment can no longer be mobilized and removed from the hole).

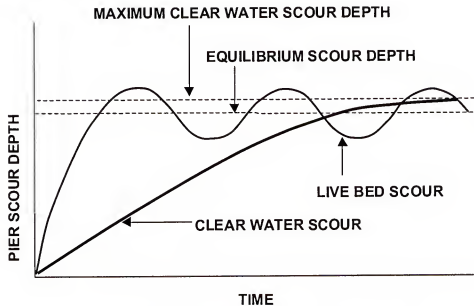


Figure 2-2. Clear water and live bed scour depth time history comparison

When sediment is in motion on the bed away from the structure, the process is known as live bed scour. In this case, the bed shear stresses away from the structure are greater than the critical value required to mobilize and transport the sediment. Generally,

initial scour rates tend to be greater in live bed scour than in clear water scour and equilibrium scour depth is attained more quickly. Under live bed scour conditions, sediment from upstream of the structure is continuously transported into the scour hole. In this case, the equilibrium condition is reached when the amount of sediment entering the scour hole is equal to the amount being removed. Under live bed scour conditions, bed forms occur and propagate through the scour hole. Thus the scour hole depth will fluctuate in time even after the "equilibrium" condition has been reached. Figure 2-2 compares typical clear water and live bed scour time series¹.

The parameter used to determine the scour regime (i.e., clear water scour or live bed scour) is the ratio of the upstream velocity to the threshold or sediment critical velocity required to cause sediment movement in the bed. This ratio, known as the flow intensity, may take one of two forms, depending on the velocity used. If the shear or bed friction velocity (u_*) is used, the ratio becomes u_*/u_{*c} . The shear velocity (u_*) is defined as $u_* = \sqrt{\tau/\rho}$, where τ is the bed shear stress. The threshold or critical shear velocity, u_{*c} , corresponds to the critical shear stress (τ_c). In this form, the flow intensity can be thought of as a shear stress ratio where $\tau/\tau_c = (u_*/u_{*c})^2$. Therefore, this form has a direct correlation to the sediment transport, since most transport equations are in terms of bed shear stress. The critical shear velocity can be determined for a given sediment (Figure 2-3), however the value of u_* is usually not readily available for prototype flow situations and must be derived using velocity profile assumptions.

The second, more common form of the flow intensity uses the depth averaged approach velocity (V) and the critical depth averaged approach velocity (V_c). The critical depth averaged approach velocity is the minimum depth averaged velocity of the

approach flow for which sediment motion will occur (for a given sediment). This form of the flow intensity (V/V_c) requires that a vertical velocity profile be known or assumed (usually logarithmic) to calculate the critical depth averaged velocity (V_c) from Figure 2-3 for a given sediment.

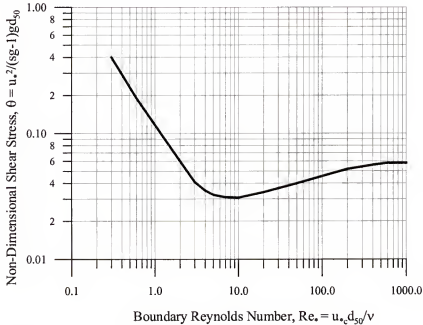


Figure 2-3. Shields' diagram

Generally, by definition clear water scour occurs when $0.5 \leq V/V_c < 1$ and live bed scour occurs when $V/V_c \geq 1$. Most researchers place the lower limit for clear water scour between $V/V_c = 0.4$ and 0.5 ; Sheppard (2002b) uses a value of 0.47 .

Melville (1975) described the stages of scour in terms of the mechanisms of scour at each phase:

- accelerated flow due to distortion of the streamlines by the cylinder,
- flow separation and horseshoe vortex development and intensification as the hole develops, and
- avalanching (or sliding) of material down the sides of the scour hole once the hole is large enough to contain the horseshoe vortex.

Melville found that the angle of the sloped sides of the hole was the angle of repose of the

sediment. This angle did not change as the scour hole grew.

Nakagawa and Suzuki (1975) divided scour into four stages:

- scour near the side of the pier caused by the tractive force of the main flow,
- scour near the leading edge of the pier generated by a horseshoe vortex,
- scour developed by the stable vortex flowing along the pier, and
- a period of reduction in scour rate due to the decrease in transport capacity in the hole.

Ettema (1980) expanded on the stages described by Melville, explaining them as

- **Initial phase:** where the scour hole forms from the flat-bed condition. In this phase, the flat-bed sediment transport begins at the sides of the cylinder and works its way around its circumference to form a shallow scour hole. He emphasized that the horseshoe vortex plays no important role in this phase.
- **Principle erosion phase:** where the horseshoe vortex grows rapidly in size and strength and settles into the scour hole resulting in an increase in the strength of the downflow at the leading edge of the cylinder. During this phase, Ettema observed that erosion occurred in a near flat, narrow strip around the pile (extending to $\pm 110^\circ$) that he called the entrainment zone. According to Ettema, as the hole develops the strength of the horseshoe vortex and the downflow weaken and eventually they are no longer able to remove sediment from the entrainment zone.
- **Equilibrium phase:** the flow is no longer able to remove sediment for clear water scour or the sediment removed is balanced by the sediment entering the hole from upstream for live bed scour.

Each of these authors and most other researchers agree on the general process of scour hole development, while differing on some of the details. Generally, for a single

cylindrical pile, there is an initial stage during which sediment is removed from the sides of the pile where the flow has been accelerated as predicted by potential flow theory. At this point, the scour resembles sediment transport on a flat bed. Ettema (1980) and Hjorth (1975) found that the points of initiation of scour occur at roughly ± 45 degrees from the leading edge of the pile. Melville (1975) found the maximum shear stress at the unscoured bed at ± 100 degrees.

As the scour hole grows, it spreads toward the leading edge of the pile, where the downflow has been established and the horseshoe vortex originates. The downflow and horseshoe vortex loosen and mobilize the sediment; and the mobilized sediment is carried downstream of the pile by the horseshoe vortex and the main channel flow. Many researchers (Shen et al. 1965, Chiew 1984) identified the downflow as the main cause of local scour and describe the scour hole as an inverted right circular cone with the pile as its axis. The angle that the slope of the hole makes with the vertical is approximately equal to the angle of repose of the sediment.

Melville (1975) observed that the horseshoe vortex is initially small and relatively weak. He conducted hot wire anemometer measurements of the downflow at the leading edge of a pile showing the velocity of the fluid entering the hole increases as the hole deepens. As the hole deepens, the downflow strengthens; and the horseshoe vortex grows and descends into the hole. At this point, Melville's measurements indicated that the circulation associated with the horseshoe vortex continues to increase, but at a decreasing rate, as the cross-sectional area of the hole increases and equilibrium is approached.

Avalanching of material down the sides of the scour hole occurs once the hole is

large enough to contain the horseshoe vortex, causing the hole to widen. Many researchers (Nakagawa and Suzuki 1975, Hjorth 1977, and Ettema 1980) observed that the actual mobilization and removal of sediment is limited to an area immediately adjacent to the pile at the base of the scour hole. As material slides down the sides of the hole, it is mobilized in this erosion or entrainment zone. Once a sediment particle is put into suspension, it is swept downstream. If the particle is carried high enough, it may interact with the wake vortices, be carried higher into the water column, and be transported and deposited well downstream. Otherwise, the sediment particle may be swept directly behind the pile and enter the relatively calm wake region between the shedding vortices. Here it will settle out and form the characteristic mound behind the structure.

Eventually, the hole becomes deep enough that the strength of the horseshoe vortex and the downflow are weakened and can no longer mobilize and remove sediment from the scour hole. In the live bed case, the amount of sediment removed is exactly equal to that deposited in the hole from upstream. At this point an equilibrium is established and the hole has reached its maximum size for the given flow conditions.

The depth of local scour is a function of a number of parameters, many of which are interrelated. For a single circular cylinder in an erodible bed, the factors influencing local scour can be divided into those describing the fluid, those describing the sediment, those describing the flow and those describing the structure.

The parameters describing the fluid are fluid mass density (ρ) and fluid viscosity (μ) or kinematic fluid viscosity ($\nu = \mu/\rho$), both of which are dependent on fluid temperature and salinity. The parameters describing the sediment are:

- characteristic grain diameter (d or d_g) which may be the median grain diameter (d_{50}) or an equivalent grain diameter (d_e) based on the grain distribution,
- standard deviation of the grain size distribution ($\sigma_g = \sqrt{d_{84}/d_{16}}$),
- density of the sediment (ρ_s),
- fall velocity (w_f), and
- angle of repose of the sediment (ϕ).

The parameters describing the flow are:

- depth of the approach flow (y_0),
- depth averaged velocity of the approach flow (V),
- roughness of the approach flow (k_s),
- energy slope of the flow (S_0),
- bed slope (S_b), and
- bed shear stress (τ).

The parameters describing the structure are the shape, dimensions, orientation relative to the flow and surface roughness. In the case of a circular pile this reduces to the pile diameter (D) and the pile surface roughness (k_c). In addition, there are derived parameters combining two or more of the above parameters, for example the critical friction or shear velocity (u_*) and the critical depth averaged velocity of the approach flow (V_c) to form the ratio V/V_c .

Dimensional analysis has been performed on these parameters to identify the dimensionless groups affecting local scour (Breusers et al. 1977, Melville and Sutherland 1989). These parameters include V/V_c , VD/v , $V^2/(gd_{50})$, $V^2/(gD)$, y_0/D , ρ_s/ρ , D/d_{50} , σ_g , and parameters describing pier shape and alignment to the flow. Shen et al. (1965, 1966,

1969) and others found a strong dependence on Froude number based on depth averaged approach velocity and flow depth ($V/\sqrt{gy_0}$). Ettema et al. (1998) discussed the use of $V^2/(gD)$ to describe the variation of flow gradients around different sized piers.

Ontowirjo (1994) examined a number of dimensionless groups with respect to equilibrium scour depth. He analyzed data from the literature and from in-house investigations. He confirmed the findings of a number of other researchers in identifying the three most influential dimensionless groups: flow intensity (V/V_c), flow aspect ratio (y_0/D) and normalized sediment size (D/d_{50}). Other researchers (Ettema et al. 1998, Melville and Chiew 1999) have determined that the same dimensionless groups are of primary importance in the temporal development of scour.

2.2 Review of Time Dependent Scour Literature

Laursen (1963) investigated the relationship of clear water scour in a long contraction as a function of geometry, flow, and sediment. He based his model on the assumption that the limit of clear water scour occurred when the boundary shear stress (the active tractive force) as a function of time (τ) was equal to the critical tractive force (τ_c). He developed an equation for the equilibrium depth of scour (d_{sc}) for a pile or abutment given by

$$\frac{d_{sc}}{y_0} = 0.8 \frac{D/2}{y_0} \left(\frac{\tau_0}{\tau_c} \right)^{2/3} \quad (2-1)$$

where y_0 is the depth of the approach flow, D is the diameter of the pile (or the length of the abutment) and τ_0 is the boundary shear stress away from the pile. From this he derived an equation for the scour hole depth as a function of time (d_s) in terms of the tractive force as a function of time

$$\frac{\tau}{\tau_c} = \left(\frac{d_{sc}}{d_s} \right)^{3/2} \quad (2-2)$$

Assuming the scour hole to be a cone with base radius of $2.75d_s$ and a height of d_s , the volume of the scour hole (V) can be approximated as $V = 8d_s^3$ and the rate of transport of material out of the scour hole is

$$q_s = \frac{dV}{dt} = 24d_s^2 \frac{dd_s}{dt} \quad (2-3)$$

where q_s is the bulk volume rate of sediment transport. He then used a simplified bedload sediment transport relationship he had previously developed (Laursen 1958):

$$c = 8 \left(\frac{d_m}{y_0} \right)^{7/6} \left(\frac{\tau}{\tau_c} - 1 \right) \quad (2-4)$$

where c in the sediment load in percent by weight and d_m is the mean sediment diameter. He assumed that the discharge per unit width was $q = V_0 y_0$, where V_0 is the depth averaged approach flow velocity, to obtain q_s . Finally, he derived a differential equation for the development of scour hole with time. For a circular pile, the equation reduces to

$$\frac{y^{3.5}}{1 - y^{1.5}} dy = 1.92 \sqrt{\frac{g}{D}} \left(\frac{d_m}{D} \right)^2 \left(\frac{\tau_0}{\tau_c} \right)^{-1.5} dt \quad (2-5)$$

where $y = d_g/d_{se}$. Laursen noted that many of his assumptions may have been questionable and warned against using his model for anything more than speculation. His main purpose was to show that "a credible prediction of the time history of scour is a reasonable hope" (Laursen, 1963, p. 116).

Note that there is no depth dependence in Eq. 2-5. This may be because Laursen is dealing directly with shear stresses on the bed, implying that the depth dependence seen by many researchers is exclusively a result of the conversion of depth averaged

velocity data to bed velocity and shear data. However, much has been made of the effect of the counter-rotating surface roller to reduce the strength of the horseshoe vortex under shallow conditions. This effect is completely neglected in Laursen's model. Also, the model does not do well for some values of d_s/d_{se} (i.e., an infinite rate of scour at $t = 0$ and $d_s = 0$).

Shen et al. (1966) conducted 21 experiments using a single cylinder diameter and sediment size, but varying the hydraulic conditions (water depth and the depth averaged flow velocity) to include both clear water and live bed conditions. From their experiments, they developed the following empirical equation for scour depth as a function of time for a pile of diameter D , in a flow with a depth averaged velocity V and an upstream water depth y_0 ,

$$\frac{d_s}{d_{se}} = 1 - e^{-aE^2} \quad (2-6)$$

The coefficients in Eq. 2-6 are determined by

$$E = \left(F \frac{D}{y_0} \right)^{1/3} \ln \left(\frac{Vt}{y_0} \right) \quad \text{and} \quad (2-7)$$

$$a = 0.026 \exp[(2.932 \text{ ft}) y_0] \quad (2-8)$$

where F is the Froude number ($F = V/\sqrt{gy_0}$).

Cunha (1975) states that the Shen et al. model was based on a narrow range of flow and sediment conditions; and therefore is probably not very well suited to practical applications. The complete absence of sediment size as a factor in the model makes it difficult to any application of the model to conditions other than those in his experiments. Note also that the model has a single time scale. Generally, experiments

examining scour time histories indicate that multiple time scales exist.

Approximating the geometry of the scour hole as the inverted frustum of a right circular cone, Carstens (1966) derived a model with a sediment balance between the geometry of the deepening scour hole and the sediment transported out of the hole. This balance gives a sediment transport rate (q_s) of

$$q_s = \frac{dV}{dt} = \frac{d}{dt} \left[\frac{\pi}{3 \tan \phi} \left(\frac{d_s^3}{\tan \phi} + \frac{3Dd_s^2}{2} \right) \right] = \frac{d_s \pi}{\tan \phi} \left(\frac{d_s}{\tan \phi} + D \right) \frac{dd_s}{dt} \quad (2-9)$$

where V is the volume of the scour hole, d_s is the depth of the scour hole and ϕ is the angle of repose of the sediment. Based on experiments by LeFeuvre (1965), and using the data of Chabert and Engeldinger (1956) to calibrate his model, Carstens empirically found the sediment transport function to be

$$q_s = 1.3 \times 10^{-5} (N_s^2 - N_{sc}^2)^{5/2} \left(\frac{d_s}{D} \right)^{-3} \left[V \left(D + \frac{2d_s}{\tan \phi} \right) d_g \right] \quad (2-10)$$

where V is the depth averaged velocity of the approach flow, d_g is a characteristic grain size, N_s is the sediment number defined as $V/\sqrt{(sg-1)gd_g}$, N_{sc} is the sediment number corresponding to the lowest value for which scour will occur, and sg is the specific gravity of the sediment. Combining Eq. 2-8 and 2-9 and integrating with respect to Vt/D gives

$$\begin{aligned} 4.14 \times 10^{-6} (N_s^2 - N_{sc}^2)^{5/2} \left(\frac{d_g}{D} \right) \left(\frac{Vt}{D} \right) &= \frac{\left(\frac{d_s}{D} \right)^5}{\tan \phi} + \frac{\left(\frac{d_s}{D} \right)^4}{16} - \frac{\tan \phi \left(\frac{d_s}{D} \right)^3}{24} \\ &+ \frac{\tan \phi \left(\frac{d_s}{D} \right)^2}{32} - \frac{(\tan \phi)^3 \left(\frac{d_s}{D} \right)}{32} + \frac{(\tan \phi)^4}{64} \ln \left(\frac{2 \frac{d_s}{D}}{\tan \phi} + 1 \right) \end{aligned} \quad (2-11)$$

Note that Eq. 2-10 does not lead to an equilibrium scour depth. Rather, the scour hole continues to grow with time. However, the rate of increase is greatly reduced and (according to Eq. 2-10) is nearly inversely proportional to $(d_s/D)^4$ at large values of d_s/D .

Carstens added a sediment inflow $[q_s(\text{in})]$ to Eq. 2-8 to account for the live bed scour case, where sediment is carried into the scour hole from upstream. With this sediment inflow, Eq. 2-8 becomes

$$q_s(\text{out}) - q_s(\text{in}) = \frac{dV}{dt} \quad (2-12)$$

Using the same bedload transport formula of Eq. 2-9 to find the rate of sediment transport into the scour hole, Carstens derives the equation

$$\begin{aligned} \frac{d_s \pi}{\tan \phi} \left(\frac{d_s}{\tan \phi} + D \right) \frac{dd_s}{dt} = 1.3 \times 10^{-5} (N_s^2 - N_{sc1}^2)^{5/2} V d_g \left(D + \frac{2d_s}{\tan \phi} \right) \left(\frac{d_s}{D} \right)^{-3} \\ - \frac{4.14 \times 10^{-6} (N_s^2 - N_{sc}^2)^{5/2}}{p} V d_g \left(D + \frac{2d_s}{\tan \phi} \right) \end{aligned} \quad (2-13)$$

where p is the porosity of the bed material (which Carstens approximates as 0.5). He states that the "sediment number, N_s , for the pile and for the bed would be identical. However, the zero transport sediment number, N_{sc1} , for the pile would be less than N_{sc2} of the bed because of higher velocities around the sides of the pile" (Carstens, 1966, p. 30). Using potential flow theory, Carstens approximates that $N_{sc2} = 2N_{sc1}$ since "the velocity at the sides of the cylinder is twice the approach velocity" (Carstens 1966, p. 30). In the live bed case, the equilibrium scour depth can be calculated.

Some discussion of Carstens' sediment number is required. Carstens originally defines N_s as $v/\sqrt{(sg-1)gd_g}$. The velocity (v) is the reference velocity; and Carstens initially defines it as the velocity adjacent to the bed. Later, he redefines v as the mean

velocity in the mainflow section (i.e., V as described above). This definition leads to a problem in determining N_s and N_{sc} since by definition $N_{sc} < N_s$, but the clear water scour regime requires $V < V_c$ and hence $N_s < N_{sc}$. This clearly cannot be the case in Eq. 2-9.

For his model development, Carstens experimentally determined N_{sc} for a given bed material, which limits the usefulness of this model to known conditions. It is possible that N_{sc} can be estimated by assuming that the zero transport condition occurs at some minimum value $V/V_c \sim \varepsilon$ and calculating N_{sc} using $V_c' = \varepsilon V_c$, where V_c would be the open channel, flat-bed depth averaged critical velocity calculated from Figure 2-3 and calculating V_c from u_{*c} by assuming a logarithmic velocity profile.

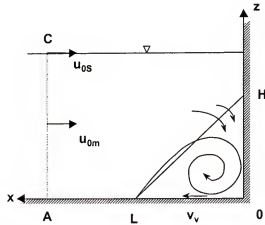


Figure 2-4. Definition sketch of the vortex in the stagnation plane

Nakagawa and Suzuki (1975) conducted experiments on scour around a vertical cylinder and concluded that the maximum scour occurred in the stagnation plane and is only dependent on the horseshoe vortex. Using the methodology of Shen et al. (1966, 1969), they modeled the vortex strength as a decrease in the circulation of the approach flow, Γ_1 , due to the presence of the pier. Assuming no slip on the solid boundaries, and using ideal fluid theory, Nakagawa and Suzuki found that the decrease in circulation could be found by $\Gamma_1 = -\frac{1}{2} u_{0s} D$, where u_{0s} is the surface velocity of the approach flow

and D is the diameter of the pier.

The circulation in the triangle H0L in Figure 2-4 was found to be $\Gamma_2 = -2\pi a v_v$, where a is the radius of rotation of the vortex and v_v is the tangential velocity at the circumference of the vortex. Through dye-injection experiments, Nakagawa and Suzuki found that vortex core lengths, L/D and H/D , had nearly constant values regardless of pier Reynolds number. Therefore, the radius of rotation of the vortex (a) became $0.177D$.

They equated the two circulations, assumed a logarithmic velocity profile and used Manning's formula to derive a formula for the tangential vortex velocity,

$$v_v = 0.45u_{m0} \left(1 + \frac{n\sqrt{g}}{\kappa y_0^{1/6}} \right) \quad (2-14)$$

where u_{m0} is the mean approach flow velocity, n is Manning's roughness coefficient, κ is von Karman's constant and y_0 is the flow depth. They assumed a drag coefficient ($C_D = 0.4$) and calculated the nondimensional bed shear stress (θ) based on the circulation of the horseshoe vortex,

$$\theta = \frac{u_*^2}{(sg-1)gd} = 0.025 \left(1 + \frac{n\sqrt{g}}{\kappa y_0^{1/6}} \right)^2 \left(\frac{C_D \alpha}{2} \right) \frac{u_{m0}^2}{(sg-1)gd} \quad (2-15)$$

where d is the representative sediment size, u_* is the shear velocity, sg is the specific gravity of the sediment, and α is a proportionality coefficient such that the projected area of the particle on the plane perpendicular to the flow is $\alpha\pi d^2/4$ and equal to approximately 0.7.

Using a stochastic sediment transport equation developed by Paintal (1971), they related the bed shear stress based on the circulation of the horseshoe vortex to the sediment transport. Paintal defined the probability that a particle starts moving in one

second as p_s , the probability that a particle comes to rest after traveling a distance λ as p_d , the area the particle occupies at the bed as $A_1 d^2$ and the volume of the particle as $A_2 d^3$. Nakagawa and Suzuki derived the rate of bed load sediment transport, q_b , by integrating the equation of continuity between x and $x + \lambda$ while holding p_d constant, where x is measured from the pile in the upstream direction (Figure 2-4). If sediment motion is limited to the stagnation plane, $x = 0$, the rate of sediment transport was found to be

$$q_b = \exp\left(-\frac{p_d}{\lambda} x\right) \int_0^x \left(\frac{A_2}{A_1} d \cdot p_s\right) \exp\left(-\frac{p_d}{\lambda} x\right) dx = \frac{A_2}{A_1} d \cdot p_s. \quad (2-16)$$

This transport or pick-up rate was assumed to be constant with scour hole depth since the "strength of the vortex is invariable during scour" (Nakagawa and Suzuki 1975, p. 232).

Paintal (1971) examined the effect of a normally distributed turbulent bed shear stress (θ) on particle motion and obtained a mean probability of movement of a particle (p_0) as a function of the mean shear stress (θ_0). For lower values of shear stress, he found that p_0 "approached a variation with the fourth power of θ_0 " (Paintal 1971, p. 537) and that for higher values of shear stress, p_0 had a limit of 0.68. He related p_0 to p_s by a characteristic time, t_0 , which is the "time through which the forces act on the particle" or the time the particle needs to escape from the bed. Paintal gave the following equation relating p_0 and p_s ,

$$t_0 = \frac{p_0}{p_s} = \frac{C_1 d}{p_0 u_*} \quad (2-17)$$

where C_1 is a proportionality constant and u_* is the shear velocity.

Nakagawa and Suzuki converted this equation to

$$\frac{p_0}{p_s} = \frac{C_1}{0.816} \sqrt{\frac{d}{(sg-1)g}} \quad (2-18)$$

and used Paintal's work to relate p_0 and θ_0 . They conducted experiments to verify his relationship. Their experiments found that $C_1 = 0.204$ in Eq. 2-18, which gave them the required relationship for p_s .

They observed that "the scour hole around a circular cylinder exhibits a definite characteristic that the deepest part along the pier has a constant width to be equal to $k_1 D$ where k_1 is an empirical constant, because the length scale of a horseshoe vortex depends only upon the pier diameter" (Nakagawa and Suzuki 1975, p. 232). Sediment on the sloped sides of the hole slides into the hole keeping the sides at the angle of repose. This sediment is removed from the region of constant width immediately adjacent to the pier. They related the rate of transport to the rate of sliding to produce a model of the time dependent scour. The rate of sediment supplied to the hole by sliding, q_{slide} , was

$$q_{slide} = \left[\frac{(1 + 2k_1)d_s}{(1 + k_1)\sin\phi} + \frac{d_s^2}{(1 + k_1)D \tan\phi \sin\phi} \right] \frac{\partial d_s}{\partial t} = S_1 \cos\phi \frac{\partial d_s}{\partial t} \quad (2-19)$$

where S_1 is the sloping area per unit length of an arc around a pier with diameter D .

Equating Eq. 2-16 and 2-19 and nondimensionalizing time by multiplying by $\sqrt{d/((sg-1)g)}$ and the scour depth by dividing by D gives

$$\left(k_1 + \frac{S_1}{D} \cos\phi \right) \frac{d(d_s/D)}{d(t/\sqrt{d/((sg-1)g)})} + \left(\frac{\varepsilon}{1 - \lambda_v} \right) \frac{A_2}{A_1} \frac{d}{D} p_s \sqrt{\frac{d}{(sg-1)g}} = 0 \quad (2-20)$$

where λ_v is the porosity and ε is the rate at which the picked particles are transported outside the hole. Integrating Eq. 2-20 under the boundary condition $d_s = 0$ at $t = 0$ gives

$$\left(\frac{k_1}{1 - \lambda_v} \right) \frac{A_2}{A_1} \frac{d}{D} p_s t = \frac{d_s}{D} + \frac{1 + 2k_1}{2(1 + k_1)\tan\phi} \left(\frac{d_s}{D} \right)^2 + \frac{\varepsilon}{3(1 + k_1)\tan^2\phi} \left(\frac{d_s}{D} \right)^3. \quad (2-21)$$

Nakagawa and Suzuki chose the coefficient values to be $k_1 = 0.25$, $\lambda_v = 0.4$, $\tan\phi = 1$,

$\varepsilon = 1.0$, $A_1 = \pi/4$ and $A_2 = \pi/6$.

While they concluded that the above model showed good agreement with experiment during the early stage of scour, Nakagawa and Suzuki admit that the model did not sufficiently describe the final stage when the rate is rapidly reduced. They attributed this to neglecting a decrease in transport capacity in the scour hole. In addition, according to Melville (1975) and Ettema (1980), the assumption of invariable vortex strength may be suspect.

Hjorth (1977) applied the sediment pick-up model originally proposed by Einstein (1950) to examine the stochastic nature of local scour. He approximated the shear stress at the bottom of the scour hole as a function of the scour depth and derived its value from a series of vorticity conservation considerations.

He assumed the magnitude of the vorticity along any vortex tube varies inversely with the cross-sectional area of the tube. Given that a vortex tube has a cross-sectional area (A) in the undisturbed flow and a different area (A') in the stagnation plane containing the horseshoe vortex, then the ratio of the magnitude of the vorticity is A/A' . He assumed that the shear stress is proportional to vorticity and that the same ratio applies. Then, he assumed that scour commences when the velocity of the approach flow is half the critical value independently of pile diameter and $A = 4A'$. Finally, he assumed that as the hole grows, the area of the horseshoe vortex grows from A' to $A' + A_h$, where A_h is the cross-sectional area of the hole (i.e., the vortex grows to fill the hole). Therefore, the ratio of the shear stress in the bottom of the scour hole to that of the undisturbed flow is $A'/(A' + A_h)$, and this ratio decreases as the hole size increases.

As Carstens and Nakagawa and Suzuki, Hjorth assumed that the hole can be

described as an inverted cone with a flat section of width $0.25D$. The cross-sectional area of the hole is then

$$A_h = 0.5 \left(\frac{D}{2} \right)^2 \left[\frac{2d_s}{D} + \left(\frac{2d_s}{D} \right)^2 \cot \phi \right]. \quad (2-22)$$

The model makes use of a stochastic bed load model that assumes that the probability of particle motion for both the approach flow and for the bottom of the scour hole is defined as the probability that for a specific particle to turbulent eddy contact, the erosive strength of the eddy is greater than the resistive force of the particle. The model also assumes that the lift on the particle is negligible and the distribution of entrainment is Gaussian. Both of these assumptions were shown by Christensen (1975) to be erroneous and though Hjorth's model does not perform well, it reproduced changes in time scales for different conditions and a smooth approach towards an equilibrium value.

Baker (1978) examined the flow of air and water around circular cylinders paying particular attention to the effects of the horseshoe vortex. He conducted several experiments to investigate the dependencies of the equilibrium scour depth and the temporal development of the scour hole on the various parameters. His scale was small with D/d_{50} ranging from 56 to 113. Cylinder diameters ranged from 2.5 to 5 cm. He used a single water depth of 10.7 cm and a median sand diameter of 0.45 mm. He measured water velocity by timing a surface drogue as it floated down a 2 meter length of his flume.

Baker verified Melville's work, observing that the deepest part of the scour hole was on the upstream stagnation line, that the upstream scour hole slope was at the angle of repose of the sand and that scouring takes place in a small, flat region close to the

cylinder. He noted that the horseshoe vortex lies in this region and stated that "during the scouring process sand is picked up by the horseshoe vortex system in this region and is carried around the cylinder and deposited in the wake. More sand then slips down the upstream surface of the scour hole into the horseshoe vortex system and the scour hole continues to grow until the shear stress beneath the horseshoe vortex system falls to such a value that it cannot remove any sediment" (Baker 1978, p. 108) or the rate of sediment entering the hole from upstream matches the rate of removal by the horseshoe vortex system.

Baker also described the development of the scour hole with time and stated that, under clear water conditions, the scour hole depth approached equilibrium slower for increasing velocities, but approached equilibrium faster for increasing velocities under live bed conditions.

Ettema (1980) developed a model along lines similar to Carstens and Nakagawa and Suzuki. However, he modeled only the upstream half of the scour hole as an inverted frustum of a right circular cone. He found that the width of the erosion area at the bottom of the scour hole, which he called the entrainment zone, varied according to D/d . For $D/d > 130$, the entrainment zone extended up the slope of the scour hole in the initial stage of scour but receded as scour progressed until it was restricted to a groove around the base of the pier. For $130 > D/d \geq 30$, particles were primarily entrained from the groove formed around the base of the pier.

Ettema derived the volume rate of change of the scour hole normalized by the cube of the pier diameter, D , as

$$\frac{\partial(V/D^3)}{\partial t} = \frac{\pi}{2} \left(\frac{d_s}{D} \cot \phi \right) \left(1 + \frac{d_s}{D} \cot \phi \right) \frac{\partial d_s}{\partial t}. \quad (2-23)$$

He defined the rate of sediment transport as

$$q_s = \sum \left(\frac{\text{number of}}{\text{particles entrained}} \right) \times \left(\frac{\text{volume per}}{\text{particle}} \right) \times \left(\frac{\text{probability for}}{\text{particle entrainment}} \right) \quad (2-24)$$

$$= \left(\frac{k_1 d}{k_2} \right) K I_p$$

where A is the surface area of the sloping sides of the upstream half of the scour hole, k_1 is a volume constant such that the volume of a sediment particle is $k_1 d^3$ (for spherical particles $k_1 = \pi/6$), k_2 is an area constant such that the projected area of a sediment particle is $k_2 d^2$ (for spherical particles $k_2 = \pi/4$) and I_p is the integral of the probability function for the entrainment of particles within the scour hole. The value of K is based on the number of particles on the surface of the entrainment zone; $N_E = k_3 A/k_2 d^2$ if the area of the entrainment zone is a function of scour depth and pier diameter or $N_E = k_4 D^2/k_2 d^2$ if the area of the entrainment zone is proportional to the pier diameter only, where k_3 and k_4 are area constants. Therefore, $K = k_3 D^3$ in the depth-and-pier diameter case and $K = A/k_4 D^2$ in the pier-only case.

Ettema's equation for the time rate of scour then becomes

$$\frac{\partial(d_s/D)}{\partial t} = \frac{k_1 d}{k_2 k_4 D^3} \frac{\sqrt{1 + \cot^2 \phi}}{\cot \phi} I_p. \quad (2-25)$$

However, Ettema was not attempting to solve for the scour depth as a function of time and so he did not attempt to evaluate I_p . He did note that I_p can be considered independent of particle size for similar values of d_s/D , flow intensity based on shear velocities (u/u_{*c}) and normalized sediment size (D/d).

Yanmaz and Altinbilek (1991) applied a sediment pick-up function to balance the inverted cone geometry of the growing scour hole with a sediment transport formula.

Using Carstens' sediment balance (Eq. 2-8), they based their sediment transport model on LeFeuvre et al. (1970) with a sediment pickup function, F_1 , of the form

$$\frac{u_s}{u} = 0.001F_1 \quad (2-26)$$

$$\text{and } F_1 = \left(\frac{C_D N_s^2}{8.2 \cos \alpha} - \tan \alpha - \tan \phi \right)^{2.5} \quad (2-27)$$

where u_s is the volume rate of bed removal per unit area of bed per unit time, u is the mean flow velocity at the particle level and α is the bed slope. Therefore, the sediment transport out of the scour hole becomes

$$\frac{q_s}{ud_{50} \left(\frac{2d_s}{\tan \phi} + D \right)} = f F_1 \quad (2-28)$$

where d_{50} is the median sediment diameter and f is a coefficient of proportionality which is a function of the geometry of the scour hole, the sediment and the flow properties.

This coefficient was experimentally determined to be

$$f = \frac{30,000}{N_s^{5.8} C_D^{2.8} (\tan \phi)^{6.4} \left(\frac{ut}{d_{50}} \right)^{0.75}} \quad (2-29)$$

where N_s is the sediment number, $u/\sqrt{(sg-1)gd_g}$, and C_D is the particle drag coefficient.

Balancing Eq. 2-8, 2-28 and 2-29 gives

$$\frac{dd_s}{dt} = \frac{30,000F_1ud_{50}}{\pi N_s^{5.8} C_D^{2.8} (\tan \phi)^{5.4} \left(\frac{ut}{d_{50}} \right)^{0.75} \left(\frac{2d_s + D \tan \phi}{d_s^2 + d_s D \tan \phi} \right)} \quad (2-30)$$

which Yanmaz and Altinbilek solved numerically using a Runge-Kutta algorithm.

Yanmaz and Altinbilek (1992) discussed the shortcomings of their analysis. The

model seems only to apply in the range of experimental values on which the coefficient f was based, which they attribute to the sensitivity of the model to the drag coefficient.

Sumer et al. (1992) developed an empirical model by assuming the scour as a function of time would follow an exponential decay curve

$$d_s = d_{sc} (1 - e^{-t/T}) \quad (2-31)$$

where d_{sc} is the equilibrium scour depth and

$$T = \frac{D^2}{((sg-1)gd^3)^{1/2}} \frac{1}{2,000} \frac{y_0}{D} \left(\frac{u_*^2}{(sg-1)gd} \right)^{-2.2}. \quad (2-32)$$

Eq. 2-32 assumes that a fully developed steady flow exists, that the boundary layer extends over the depth of the flow and that the flow regime is rough. Calculation of the Reynolds number for the experiments upon which Eq. 2-32 is based indicate that the rough flow regime was not reached, therefore calculations of bed shear velocity and shear stress were likely incorrect. Also, as noted for Shen et al. (1966), the use of a single time scale is likely an over simplification.

Kothyari et al. (1992) used a methodology similar to Hjorth (1977). They approximated the shear stress at the bottom of the scour hole by assuming that the shear stress under the vortex is a function of the cross-sectional area of the vortex and that the vortex expanded to fit the scour hole. Experimentally, they found that the initial diameter of the horseshoe vortex, D_v , was

$$\frac{D_v}{y_0} = 0.28 \left(\frac{D}{y_0} \right)^{0.85}. \quad (2-33)$$

As scour progresses and the vortex descends into the scour hole, the cross-sectional area of the vortex grows from an initial value of $A_0 = \pi D_v^2/4$ to

$$A_t = \frac{\pi D_v^2}{4} + \frac{d_s^2}{2 \cot \phi} \quad (2-34)$$

where A_t is the total area of the vortex.

Based on the fact that the initiation of scour occurs at approximately one half the critical shear velocity of the undisturbed bed, Kothyari et al. proposed that the probabilistic shear stress at the nose of the pier at time t , $\tau_{p,t}$, can be determined from

$$\tau_{p,t} = 4.0 \tau_u \left(\frac{A_0}{A_t} \right)^C = \rho u_{*,t}^2 \quad (2-35)$$

where τ_u is the shear stress on the undisturbed bed, $u_{*,t}$ is the bed shear velocity at the nose of the pier at time t and C is a coefficient found experimentally to be 0.57.

Using Paintal's (1971) stochastic model for sediment transport, they calculated the time required to remove one particle, t_0 , from Eq. 2-17 and combine it with Eq. 2-35 to obtain

$$t_0 = \frac{C_1 d}{p_{0,t} u_{*,t}} \quad (2-36)$$

where $C_1 = 0.05$ by experiment and p_0 and u_* are time dependent (i.e., $p_{0,t}$ and $u_{*,t}$). They developed the relationship

$$\begin{aligned} \frac{\tau_{p,t}}{\rho g (sg - 1) d} \leq 0.25, \quad p_{0,t} &= 0.45 \left(\frac{\tau_{p,t}}{\rho g (sg - 1) d} \right)^{3.45} \\ \frac{\tau_{p,t}}{\rho g (sg - 1) d} > 0.25, \quad p_{0,t} &= 1 \end{aligned} \quad (2-37)$$

Note that their limit for p_0 is 1.0. This is greater than the 0.68 limit Paintal found for high shear stresses.

Kothyari et al. gave the following procedure for finding the scour depth as a function of time:

- compute $\tau_{p,i}$ and $d_s = 0$ with Eq. 2-35;
- calculate t_0 with Eq. 2-36;
- increment d_s by one grain diameter over time t_0 ;
- increment the time by t_0 recomputed $\tau_{p,i}$ for the incremented d_s and repeat the procedure.

Scour ends when $\tau_{p,i} \leq \tau_{cs}$, the critical shear stress from Figure 2-3.

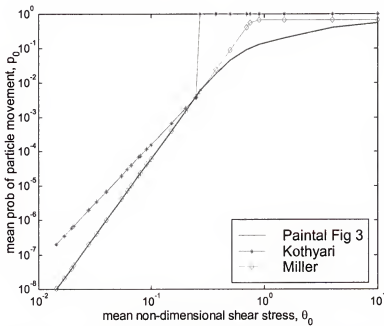


Figure 2-5. Comparison of Paintal (1971) results and approximations from Eq. 2-36 and 2-37

It should be noted that the initial vortex diameter in the Kothiyari et al. model is independent of the velocity of the flow and is only a function of the depth of the flow and size of the pier. That the size of the horseshoe vortex could be independent of the flow is unlikely. Also, the estimate Kothiyari et al. used for Paintal's mean probability of particle movement, p_0 , is not in very good agreement with Figure 3 from Paintal (1971). A better approximation would be for

$$\begin{aligned}\theta_{p,t} &\leq 0.75, \quad p_{0,t} = 2\theta_{p,t}^{4.5} \\ \theta_{p,t} &> 0.75, \quad p_{0,t} = 0.68\end{aligned}\quad (2-38)$$

where $\theta_{p,t} = \tau_{p,t} / \rho g (sg - 1) d$. Figure 2-5 compares Eq. 2-37 and 2-38 with data from Paintal (Paintal 1971, Figure 3).

To find the equilibrium scour depth for experiments stopped before equilibrium was reached, Bertoldi and Jones (1998) fitted their data using

$$d_s(t) = a \left[1 - \frac{1}{(1 + abt)} \right] + c \left[1 - \frac{1}{(1 + cdt)} \right]. \quad (2-39)$$

No attempt to relate the coefficients (a, b, c, d) to the flow and sediment parameters was reported. This equation has been found to fit experimental time series' quite well (Gosselin 1997). Bertoldi and Jones determined the coefficients for Eq. 2-39 by fitting a long-term experiment (at least 72 hours) with sediment of the same size as that of the data set to be extrapolated. An offset between the long-term and short-term data was found at the 24 hour mark and this offset was applied to the curve determined from the long-term experiment. Only sediment size and velocity were varied in their experiments. Pier effective diameter and approach flow depth were constant throughout the experiments, so their effect on this method is undetermined.

Melville and Chiew (1999) examined the time required to reach equilibrium for clear water scour conditions and determined a time scale factor for application to an equilibrium scour equation (Melville 1997), to account for the duration of a scouring event. They defined t_e as the time when the scour hole develops to a depth at which the rate of increase in depth does not exceed five percent of the pier diameter in the succeeding 24 hour period

$$\frac{dd_s(t_e)}{dt} \leq \frac{0.05D}{24 \text{ hrs}}. \quad (2-40)$$

They defined the equilibrium time scale as $t^* = Vt_e/D$, where V is depth averaged approach velocity and D the pier diameter. Experimentally, they found that t_e and therefore t^* were dependent only on V/V_c , D/d_{50} and y_0/D .

Melville and Chiew conducted experiments and combined their data with experiments by Ettema (1980). They determined the relationship between the equilibrium time scale (t^*) and each of the independent parameters. Their analysis showed that for $y_0/D > 6$ or $D/d_{50} > 100$, t^* was not dependent on y_0/D or D/d_{50} , respectively. Under these condition $t^* = 2.5 \times 10^6$ (the upper limit). Below these limiting values, they found the following relationships:

$$\frac{y_0}{D} \leq 6, \quad t^* = 1.6 \times 10^6 \left(\frac{y_0}{D} \right)^{0.25}. \quad (2-41)$$

$$\text{and } \frac{D}{d_{50}} \leq 100, \quad t^* = 9.5 \times 10^5 \left(\frac{D}{d_{50}} \right)^{0.21}. \quad (2-42)$$

In the case of D/d_{50} , Melville and Chiew stated that some of Ettema's data did not show consistent trends and so they did not recommend Eq. 2-42.

Their experiments were limited to the clear water scour range ($0.4 \leq V/V_c \leq 1$) and the dependency of t^* on V/V_c in this range was found to be

$$t^* = 4.17 \times 10^6 \left(\frac{V}{V_c} - 0.4 \right). \quad (2-43)$$

Neglecting the D/d_{50} relationship, Eq. 2-41 and 2-43 were combined and the resulting equations were solved for the time to equilibrium (t_e). For $y_0/D > 6$ the solution is only dependent on V/V_c and D/V :

$$t_e \text{ (days)} = (48.26 \text{ days/sec}) \frac{D}{V} \left(\frac{V}{V_c} - 0.4 \right). \quad (2-44)$$

For $y_0/D \leq 6$, the equation becomes

$$t_e \text{ (days)} = (30.89 \text{ days/sec}) \frac{D}{V} \left(\frac{V}{V_c} - 0.4 \right) \left(\frac{y}{D} \right)^{0.25}. \quad (2-45)$$

Melville and Chiew fitted the following equation for scour as a function of time:

$$\frac{d_s}{d_{sc}} = \exp \left[-0.03 \left| \frac{V_c}{V} \ln \left(\frac{t}{t_e} \right) \right|^{1.6} \right]. \quad (2-46)$$

From Eq. 2-44 and 2-45, the time to equilibrium (t_e) tends to increase with increasing flow intensity, V/V_c , holding the other variables constant. Melville and Chiew explained this by considering opposing effects of increasing flow intensity. First, as flow intensity increases sediment is more rapidly removed from the scour hole tending toward a shorter time to equilibrium. Second, the equilibrium scour depth tends to increase with increasing flow intensity which tends to result in a longer time to reach equilibrium. This study found that the latter effect was dominant and, as Baker (1978) found, that higher flow intensity resulted in longer times to reach equilibrium for clear water scour flow intensities.

Figures 2-6 is comparison plots of some of the models described above against a University of Florida clear water data set. Note that Carstens (1966) and Melville and Chiew (1999) both reproduce the shape of the data in Figure 2-6, however the magnitude is not matched. Kothyari et al. (1992) does not do well in the data comparison with the change in scour rate occurring much too soon. The best that can be said is that there exists a wide range of results and the models (Carstens, Kothyari et al., Melville and

Chiew, Shen et al., and Sumer) should probably not be used outside the narrow range of conditions for which they were developed.

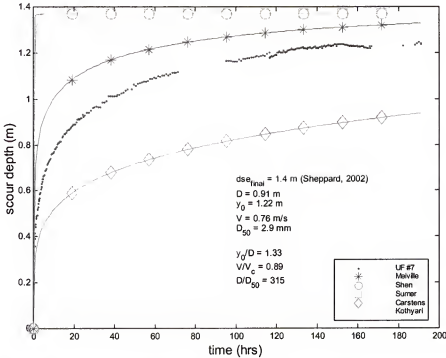


Figure 2-6. Comparison of model performance with UF data set 7.

2.3 Scour Hole Hydrodynamics and Fluid Modeling

Shen et al. (1965, 1966) conducted experiments to examine the flow fields within a scour hole. They allowed scour to occur around a six inch cylinder in a flume. They stopped the experiment at some desired depth and fixed the entire flume bed with an adhesive solution. Then, they re-established the flow and measured the velocity distributions with a small pitot tube and yarn streamers to establish direction.

With their velocity measurements, Shen et al. (1965) showed that potential flow theory can be used away from the boundary and the velocity distribution in the jet above the scour hole is $u/u_0 = (1 - y/\delta)(y/\delta)^{1/7}$, where u is the velocity at a given elevation above the bed (y), u_0 is a characteristic velocity in the boundary layer and δ is the

thickness of the boundary layer. "Theoretically, the maximum velocity in the boundary layer is equal to $0.65u_0$ " (Shen et al. 1965, p. 5). They verified similarity of the velocity profiles upstream of the cylinder and that the line of flow separation made a 7 degree angle with the horizontal.

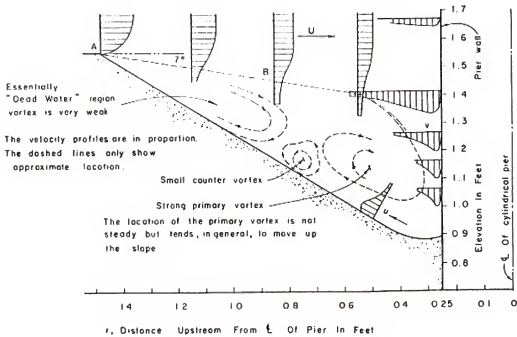


Figure 2-7. Flow field inside the scour hole (Shen et al. 1966)

Figure 2-7 shows the velocities within the scour hole measured by Shen et al. (1966). They found a low velocity region on the scour hole wall above approximately one-half the depth of scour and a clear depression in the bed at the base of the cylinder. This indicates that the majority of the sediment mobilization and entrainment occurs at the base of the pile rather than along the sides of the hole, an assumption important to many of the models discussed in the previous section.

Melville (1975) mapped the flow field in a scour hole for three different stages of the scour process during one set of conditions. The stages were:

- flat-bed or no scour stage at $t = 0$,

- intermediate stage at $t = 0.5$ hours, and
- equilibrium condition.

At each stage a cast was made of the bed, the surface of the cast was coated with sand from the experiment and the flow conditions were re-established. Flow measurements were made using a hot-film anemometer with direction indicated by a piece of cotton on the rod.

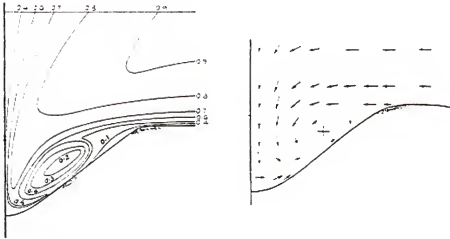


Figure 2-8. Flow field inside the scour hole for the intermediate stage (Melville 1975)

Figure 2-8 shows an example of Melville's measurements for the intermediate stage. Velocities along the sides of the hole were weak and a groove or lip exists at the base of the cylinder where velocities are highest, indicating this is the most important area of sediment removal. Melville also found that scour was initiated at points ± 100 degrees of the leading edge of the cylinder, that a strong downflow develops at the leading edge, and that the velocity at the bottom of the hole decreases as equilibrium is reached. He theorized that as the scour hole grows the diameter of the horseshoe vortex increases and the center of the vortex moves away from the cylinder. The circulation associated with the vortex increases rapidly initially and then slows and approaches a constant value as equilibrium is reached.

Dey et al. (1995) derived quasi-analytical equations for the flow field (in the scour hole, adjacent to the pier above the flat bed, and in the wake region) by satisfying the continuity equations and determining empirical coefficients (by curve fitting experimental data). They conducted clear water scour experiments using two sand diameters, three pier diameters, three approach flow depths, and six approach flow velocities. When they determined that equilibrium had been reached, the flume was drained and the bed was stabilized using a synthetic resin. The authors compared their solutions and the measurements performed by Melville (1975). The equations showed good agreement with the measurements and the authors maintain that the model may be useful for simulating the flow field under prototype conditions.

Gosselin (1997) conducted a series of clear water scour experiments to measure the velocity field (at various times during the scour process). These experiments included a circular cylinder with a diameter of 0.17 m, a median sand size of 0.17 mm, a flow depth of 0.35 m and a depth averaged velocity of 0.25 m/s. For comparison and evaluation, he also modeled the velocities in the scour hole with a commercial three-dimensional flow model.

During his experiments, Gosselin stopped the flow at 1, 6 and 24-hour intervals, drained the flume, mapped and fixed the bed. With the flow re-establishing, he measured the velocities with an acoustic Doppler velocimeter. Figure 2-9 shows a plot of the flow field obtained at 6 and 24 hours at points 0 and 60 degrees about the cylinder. Note the low velocity areas on the sloped side of the scour hole. Figure 2-10 shows the contours of the bed at 1, 6 and 24 hours. The plots show further evidence of the symmetry of the scour hole and the concentration of erosion at the base of the scour hole.

Gosselin's experiments verified the stages of scour discussed by Ettema (1980). Based on his experiments, he "hypothesized that the circulation associated with horseshoe vortex does not appear in the stagnation plane until there is separation at the upstream edge of the hole" (Gosselin 1997, p. 191). In later stages, Gosselin found that the horseshoe vortex filled the scour hole and was well defined even in the 90 degree plane. Thus, the initial stage of scour is essentially a period of flat-bed sediment transport with accelerated flows followed by a period of intense scour due to the horseshoe vortex. He also verified the suitability of using Eq. 2-39 of Bertoldi and Jones (1998) to reproduce the temporal development of the scour hole.

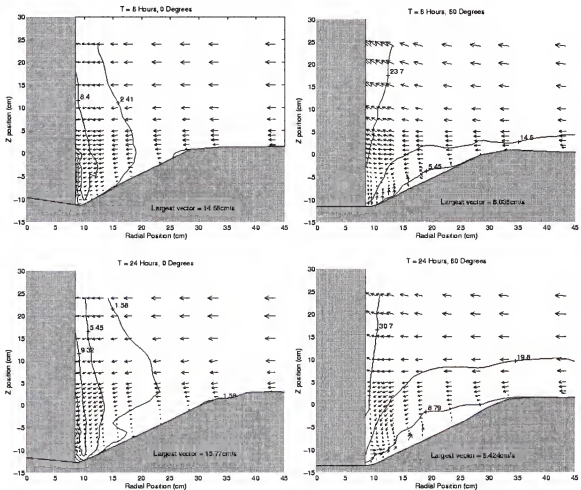


Figure 2-9. Flow fields inside the scour hole at 6 hours and 24 hours: $D = 0.17$ m, $D/d_{50} = 1000$, $y_0/D = 2$, $V/V_c = 0.9$ (Gosselin 1997)

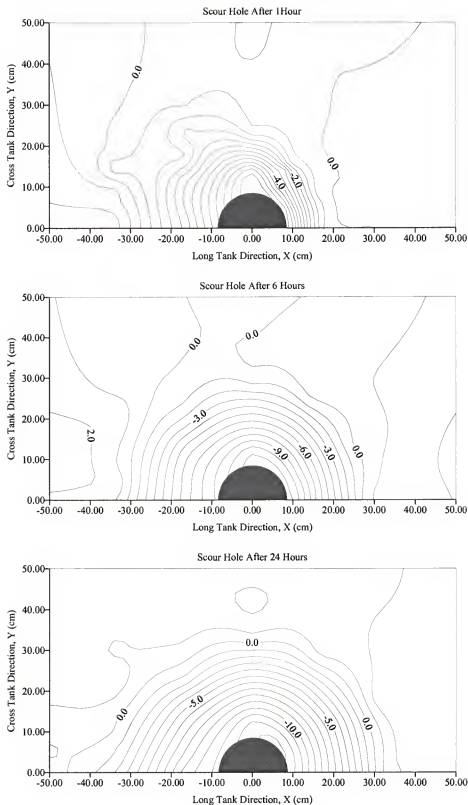


Figure 2-10. Scour hole contours at 1, 6 and 24 hours: $D = 0.17$ m, $D/d_{50} = 1000$, $y_0/D = 2$, $V/V_c = 0.9$ (Gosselin, 1997)

Ahmed and Rajaratnam (1998) conducted experiments to investigate the flow and bed shear stress around cylinders placed in different types of beds; smooth and rough, with and without a scour hole. They found that rough beds "induced a steeper pressure gradient and thus a stronger downflow" (Ahmed and Rajaratnam 1998, p. 293) and that the downflow was as much as 95% of the approach velocity with a scour hole present vice 35% without one. They defined the shear amplification as a ratio of the local shear stress to the approach flow shear stress and found it could be as much as 13.5 for rough beds.

Totapally et al. (1999) examined the temporal variations of local scour under steady flow and using stepped hydrographs. They concluded that a logarithmic equation represented the variation of scour with time better than a power equation and questioned the existence of an equilibrium depth, maintaining that scour will continue with time though at a greatly reduced rate. They examined the use of superposition to calculate the scour depth under stepped hydrographs (i.e., time series' developed from steady flow experiments are applied at each step of the hydrograph as if the step were a separate steady flow run). They found that for steps with a duration on the order of 2.5 hours, the results of the superpositioning were comparable to experiments. For shorter duration steps, the superposition method tended to under predict the measured scour depth. Totapally et al. also found the scour holes to be geometrically similar at different times in the scour time history.

Graf and Istiarto (2001) investigated the flow patterns in planes upstream and downstream of a cylinder and vertically in the scour hole using an acoustic-Doppler velocity-profiler (ADVP). They found that the shear stress was reduced in the scour hole

as compared to the approach flow but that the turbulent kinetic energy was very strong at the foot of the cylinder on the upstream side. The turbulent kinetic energy was also very strong in the wake behind the cylinder.

There have been few attempts to numerically model the flow field within a scour hole, much less couple such a hydrodynamic model with a sediment transport model to reproduce the growth of a scour hole. Wang and Jia (1999) examined the importance of including various flow effects on sediment transport. They used a numerical model to simulate the three dimensional flow conditions around a pile and in a scour hole. Empirical functions were used to alter the shear stress in an empirical sediment transport model to account for the effects of the mainflow, downflow, vorticity, and turbulence intensity on sediment transport within the scour hole. After calibrating their model with experimental data, Wang and Jia claim their model produced reasonable results.

Chang et al. (1999) used a large-eddy simulation (LES) model to solve the flow equations around a bridge pier with a fixed bed and no scour. Then, they adjusted the flat-bed shear stress to account for the bed deformation without recomputing the flow equations. They applied this adjusted shear stress to Van Rijn's (1984) bed-load formula to calculate the sediment transport and tested their results against the time series data of Ettema (1980). They found their results in good agreement with the data, supporting the method of applying flat-bed sediment transport formula with an adjusted shear stress value to model the scour hole development with time.

Sumer et al. (2002) used a finite volume hydrodynamic model with $k-\omega$ turbulence modeling to simulate the 3-D flow around a pile. They coupled this flow model with a 2-D vectorial representation of the bed-load equation of Engelund and

Fredsoe (1976) to simulate scour around the pile. Sumer et al. were able to capture all the main features of the scour process (i.e., the horseshoe vortex, sand slides or avalanching on the sides of the scour hole, bed ripples, the shape of the scour hole) and their equilibrium scour depth agreed fairly well with measurements. Conditions for the simulation were: $D = 10$ cm, $d_{50} = 0.26$ mm, depth of flow = 20 cm, $V/V_c = 1.6$.

Equilibrium was reached in approximately 2.5 hours, but computation time for the model was 2.5 months on an Alpha 21264 workstation (equivalent to a 1.5 GHz Pentium 4 PC). This makes the Sumer model impractical for prototype size calculations where the time to equilibrium is on the order of weeks.

CHAPTER 3 EXPERIMENTS AND RESULTS

3.1 Clear Water Scour Experiments

Clear water scour experiments were conducted by University of Florida researchers under the direction of Dr. D. Max Sheppard in a large, flow-through type flume located in the Conte USGS-BRD Laboratory in Turners Falls, Massachusetts from August 1998 to April 2002. The flume was 6.1 m wide, 6.4 m deep, and 38.4 m long. The data from the experiments were reduced and analyzed by the author of this dissertation and Dr. Sheppard at the University of Florida.

The experiments utilized three different circular test cylinders with diameters of 0.915, 0.305 and 0.114 m; three different sediment grain sizes with median diameters of 0.22 mm ($\sigma_g = 1.5$), 0.80 mm ($\sigma_g = 1.29$) and 2.9 mm ($\sigma_g = 1.21$) and a range of water depths and velocities. The intention was to maintain flow intensities (V/V_c) in the clear water scour range. In several cases post-experiment data analysis revealed flow intensities had crossed into the live bed range. Due to the length of the test section (9.8 m), it is believed that sufficient sand supply existed to include these experiments in the live bed analysis.

Figure 3-1 shows the physical layout of the flume. The test section was the width of the flume, 9.8 m long and started 24.4 m downstream of the entrance. The sediment in the test area was 1.83 m deep. Water for the flume was supplied from a hydroelectric power plant reservoir adjacent to the building housing the flume. Water flowed from the

reservoir, through the flume, and discharged into the Connecticut River downstream of control structures in the river. The drop in water elevation from the reservoir to the bottom of the flume was approximately 6.5 m.

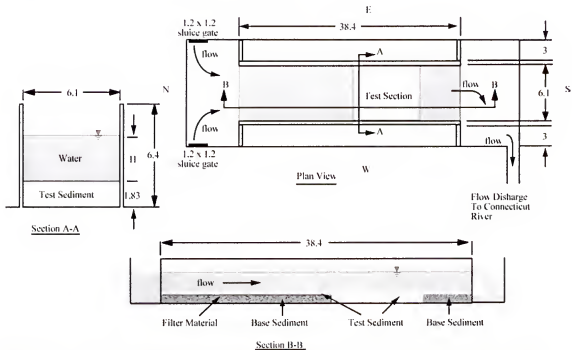


Figure 3-1. Conte USGS-BRD Laboratory flume (not to scale, all lengths in meters)

The Conte USGS-BRD Laboratory flume was chosen for these tests because of its large size. This allowed structures up to 0.9 m in diameter to be tested. The main advantage of a gravity driven flume is that large flow discharges can be obtained without pumps. A disadvantage is that other than water depth and flow velocity there is little control on the water used in the experiments. For example, the water temperature will be that of the reservoir. During the period of testing the water temperatures ranged from slightly above freezing in the winter months to around 26C in the summer.

Constituents in the water, specifically suspended fine sediment, could not be controlled. During the course of the experiments, it became clear that the level of suspended fine sediments in the water column had an impact on the equilibrium scour

depth. This was particularly true during times of heavy runoff from the Connecticut River drainage basin. The effects of suspended fine sediments are discussed in detail by Sheppard (2002a, 2002b) and Smyre (2002).

The scour hole depth was monitored by video cameras inside (and on some occasions outside) the cylinders and by arrays of acoustic transponders attached to the cylinders just below the water surface. Figure 3-2 shows the orientation of the cameras and the acoustic arrays. Data collection and instrumentation for these tests are described in Appendix D.

The tests lasted from 41 hours to 650 hours and, in most cases, near equilibrium scour depths were achieved. Following each test, the flume was drained and the scour hole topography was measured with a point gage. Since the location of the maximum scour depth did not necessarily correspond to any of the continuously monitored locations, a correction was required to correlate the scour depth time series curve to the maximum measured scour depth.

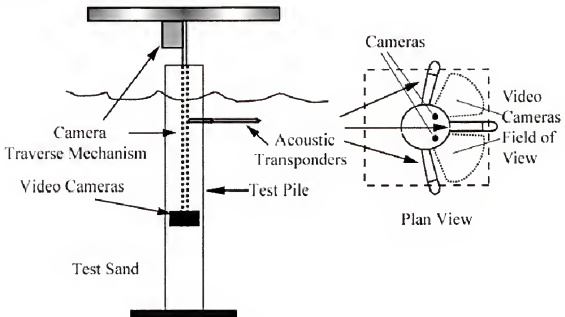


Figure 3-2. Schematic of a test cylinder with mounted acoustic arrays and cameras

3.2 Live Bed Scour Experiments

The live bed scour experiments required higher flow velocities than those achievable in the gravity driven flume at the Conte USGS-BRD Laboratory. They also required a system to provide a continuous sediment supply to the scour hole and the upstream bed. Such a flume was available in the Hydraulics Laboratory at the University of Auckland in Auckland, New Zealand.

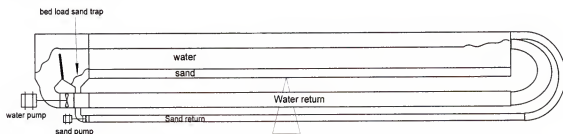
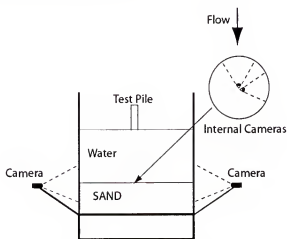


Figure 3-3. Schematic drawing of the University of Auckland Hydraulics Laboratory Sediment Recirculating Flume



Flume Cross-Section

Figure 3-4. Cross-sectional schematic of the University of Auckland Hydraulics Laboratory Sediment Recirculating Flume with test cylinder and video cameras

This flume was 1.52 m wide by 1.22 m deep by 45 m long with the ability to tilt up to 1%. Two impeller type pumps provided a maximum water discharge of 1.2 cubic meters per second and another impeller type pump provided a maximum sediment

recirculation rate of 0.06 cubic meters per second. Bed load sediment was trapped and pumped to the flume entrance with this sediment pump. Suspended sediment was pumped with the water to the entrance with either or both of the water pumps. Figures 3-3 and 3-4 are schematic drawings of this flume.

Experiments were conducted from January through April of 2002 by Dr. D. Max Sheppard and Dr. Bruce W. Melville at the University of Auckland using a single 0.152 m diameter test cylinder and sediment with median diameters of 0.27 mm ($\sigma_g = 1.32$) and 0.84 mm ($\sigma_g = 1.32$). Water depth was maintained constant throughout each experiment and ranged from 0.2 m to 0.4 m. The depth-averaged velocity was varied to achieve flow intensities (V/V_c) from 0.6 to 6.0.

As in the clear water scour tests, the scour depth was measured by both video cameras located inside the test cylinder and an array of three acoustic transponders attached to the outside of the cylinder. This arrangement and the acoustic arrays were identical to that used in the clear water scour tests. Figure 3-4 shows the orientation of the internal cameras to the water flow in the flume. Unlike the camera orientation for the clear water scour tests shown in Figure 3-2, the orientation for the live bed scour tests is asymmetric with one camera looking directly upstream and the other looking across the flow.

Since sediment suspended in the water column during the live bed test settled in the scour hole when the flow was stopped and the flume was drained, a survey of the scour hole by point gage to determine the maximum scour depth reached during the test was not done. Therefore, no topographic plots of the scour holes after the tests are available.

3.3 Experimental Results and Data Analysis

Table 3-1 lists the conditions and final scour depths for each test.

Table 3-1. Measured experimental data summary

Exp. No.	Exp. Duration (hrs)	D (m)	V (m/s)	y ₀ (m)	d ₅₀ (mm)	σ _g	T (C)	max scour depth (m)	T.S. Data Source	max T.S. depth (m)
1	89	0.114	0.29	1.19	0.22	1.51	18	0.13	video	0.12
2	163	0.305	0.31	1.19	0.22	1.51	18	0.26	video	0.26
3	360	0.914	0.40	1.27	0.80	1.29	7	1.11	video	1.00
4	143	0.914	0.39	0.87	0.80	1.29	1	0.64	video	0.61
5	88	0.305	0.39	1.27	0.80	1.29	1	0.42	pinger 1	0.35
6	41	0.114	0.41	1.27	0.80	1.29	4	0.19	pinger 1	0.14
7	188	0.914	0.76	1.22	2.90	1.21	1	1.27	video	1.25
8	330	0.914	0.65	0.56	2.90	1.21	2	1.06	video	1.02
9	448	0.914	0.57	0.29	2.90	1.21	2	0.90	video	0.90
10	616	0.914	0.50	0.17	2.90	1.21	6	0.66	video	0.66
11	350	0.914	0.70	1.90	2.90	1.21	12	1.00	video	0.95
12	256	0.305	0.40	1.22	0.22	1.51	15	0.38	video	0.38
13	258	0.305	0.30	0.18	0.22	1.51	10	0.30	video	0.28
14	460	0.914	0.30	1.81	0.22	1.51	24	0.79	pinger 1	0.79
15	291	0.914	0.32	1.95	0.22	1.51	20	0.82	pinger 1	0.82
16	646	0.914	0.44	2.00	0.22	1.51	21	1.22	video	1.19
202	5.1	0.152	0.62	0.42	0.27	1.32	22	0.24	video	0.24
203	7.4	0.152	0.88	0.43	0.27	1.32	23.5	0.34	video	0.34
204	7.8	0.152	1.10	0.40	0.27	1.32	23	0.26	video	0.26
205	4.3	0.152	1.26	0.40	0.27	1.32	23	0.30	video	0.30
207	1.7	0.152	0.55	0.20	0.27	1.32	24	0.18	video	0.18
208	25	0.152	0.69	0.43	0.27	1.32	23	0.33	pinger 5	0.33
209	46	0.152	0.25	0.49	0.27	1.32	23	0.14	video	0.14
210	19	0.152	0.37	0.43	0.84	1.32	23	0.20	video	0.20
211	51	0.152	0.58	0.38	0.84	1.32	24.5	0.19	video	0.19
212	17	0.152	0.74	0.38	0.84	1.32	24.5	0.27	video	0.27
213	16	0.152	1.05	0.38	0.84	1.32	26	0.29	video	0.29
214	1.7	0.152	1.21	0.38	0.84	1.32	23	0.29	video	0.29
215	1.0	0.152	1.37	0.38	0.84	1.32	26	0.29	video	0.29
217	1.1	0.152	1.52	0.30	0.84	1.32	23	0.31	video	0.31
222	332	0.152	0.25	0.43	0.84	1.32	18	0.16	video	0.16

Column 9 (max scour depth) of Table 3-1 is the maximum scour depth measured by point-gage following the tests. In the live bed tests, this is the maximum observed in the recorded scour depth time series (T.S.). Column 10 (T.S. Data Source) is the source of the scour depth time series used in the analysis (either video or acoustic). When the

source data is acoustic, the number of the transponder (pinger) in the array is noted (Figure 3-2). Column 11 (max T.S. depth) is the maximum scour depth recorded in this time series.

Table 3-2 summarizes standard parameters computed from the data in Table 3-1.

These parameters are required to nondimensionalize the data.

Table 3-2. Computed parameters summary

Exp. No.	ρ (kg/m ³)	ν (m ² /s)	d*	RR	k_s (mm)	u_* (m/s)	u_{*c} (m/s)	V_c (m/s)
1	998.6	1.1E-06	5.38	5	1.10	0.011	0.013	0.32
2	998.6	1.1E-06	5.38	5	1.10	0.012	0.013	0.32
3	999.9	1.4E-06	15.94	2	1.60	0.017	0.020	0.46
4	999.9	1.7E-06	14.04	2	1.60	0.017	0.020	0.45
5	999.9	1.7E-06	14.11	2	1.60	0.016	0.020	0.46
6	1000.0	1.6E-06	14.95	2	1.60	0.017	0.020	0.46
7	999.9	1.7E-06	50.77	2	5.80	0.039	0.044	0.85
8	999.9	1.7E-06	51.78	2	5.80	0.037	0.044	0.77
9	999.9	1.7E-06	51.98	2	5.80	0.036	0.044	0.70
10	999.9	1.5E-06	56.74	2	5.80	0.034	0.045	0.65
11	999.5	1.2E-06	63.43	2	5.80	0.034	0.046	0.94
12	999.1	1.1E-06	5.10	5	1.10	0.016	0.013	0.33
13	999.7	1.3E-06	4.65	5	1.10	0.015	0.014	0.27
14	997.2	9.1E-07	5.94	5	1.10	0.012	0.012	0.32
15	998.2	1.0E-06	5.56	5	1.10	0.012	0.013	0.33
16	998.0	9.8E-07	5.66	5	1.10	0.017	0.013	0.33
202	997.7	9.5E-07	7.06	5	1.35	0.029	0.013	0.28
203	997.4	9.2E-07	7.23	5	1.35	0.041	0.013	0.28
204	997.5	9.3E-07	7.17	5	1.35	0.054	0.013	0.27
205	997.5	9.3E-07	7.17	5	1.35	0.062	0.013	0.27
207	997.2	9.1E-07	7.29	5	1.35	0.028	0.013	0.25
208	997.5	9.3E-07	7.17	5	1.35	0.032	0.013	0.28
209	997.5	9.3E-07	7.17	5	1.35	0.011	0.013	0.28
210	997.5	9.3E-07	22.32	2	1.68	0.017	0.020	0.41
211	997.1	9.0E-07	22.85	2	1.68	0.027	0.020	0.41
212	997.1	9.0E-07	22.85	2	1.68	0.035	0.020	0.41
213	996.7	8.7E-07	23.37	2	1.68	0.054	0.021	0.41
214	997.5	9.3E-07	22.32	2	1.68	0.062	0.020	0.41
215	996.7	8.7E-07	23.37	2	1.68	0.070	0.021	0.41
217	997.5	9.3E-07	22.32	2	1.68	0.080	0.020	0.40
222	998.6	1.1E-06	20.54	2	1.68	0.011	0.020	0.41

Water mass density (ρ) and kinematic viscosity (ν) were assumed to be only a function of water temperature. The mass density equation used was the International

Equation of State of Sea Water, 1980 (Pond and Pickard 1983). To calculate viscosity, data tables from Kennish (1989) and Newman (1977) were fitted and the resulting equation used. These data tables and the fitted formulas are in Appendix C.

A standard laboratory convention was used for relative roughness. For non-ripple producing sediments ($d_{50} > 0.6$ mm) the relative roughness (RR) was set at 2. For ripple producing sediment ($d_{50} \leq 0.6$ mm), the relative roughness was set at 5.

The friction velocity (u_*) was calculated by assuming a logarithmic velocity profile and using the relationship between bed shear stress and depth-averaged velocity, water depth, and relative roughness for fully developed flow (Sleath 1984). The critical friction velocity was calculated using a curve fit of the Shields Diagram shown in Figure 2-3. Finally, the depth averaged critical velocity was calculated assuming a fully developed logarithmic velocity profile. These formulas are detailed in Appendix C.

Table 3-3 is a summary of the standard nondimensional parameters obtained by combining the measured data and computed parameters from Tables 3-1 and 3-2. The experiments with complete time series data covered ranges of V/V_c from 0.6 to 4.6, D/d_{50} from 140 to 4155, y_0/D from 0.2 to 11, and pier Reynolds numbers (VD/ν) from 3×10^4 to 5×10^5 . Bed Reynolds numbers ($u_* k_s/\nu$ and $u_{*c} k_s/\nu$) are included to show the flow regime for each test (most tests were in the "transition" regime of the logarithmic velocity profile). Also included are the nondimensional upstream and critical shear stress values (θ_u and θ_c).

Acoustic Time Series Data Processing

Each test produced several time series showing the scour hole development: two from the video cameras and twelve from the acoustic arrays (four for each of three

arrays). Potentially, each test had five available time histories showing the scour hole development immediately adjacent to the test cylinder which is the theoretical location of maximum scour. These time histories are designated "video" (the maximum of the two video camera records) and "pinger 1, pinger 5 or pinger 9" corresponding to the transponders closest to the test cylinder in each of the three acoustic arrays (Figure 3-2).

Table 3-3. Nondimensional parameters summary

Exp. No.	V/V_c	y_0/D	D/d_{50}	VD/v	u_*k_s/v	$u_{*c}k_s/v$	θ_u	θ_c
1	0.90	10.40	518.2	3.1E+04	12.0	13.5	0.037	0.047
2	0.96	3.90	1386.4	8.8E+04	12.8	13.5	0.042	0.047
3	0.87	1.39	1142.5	2.6E+05	18.7	21.8	0.021	0.029
4	0.86	0.95	1142.5	2.0E+05	15.7	18.1	0.022	0.030
5	0.84	4.16	381.3	6.9E+04	15.4	18.3	0.021	0.030
6	0.89	11.14	142.5	3.0E+04	17.5	19.9	0.023	0.029
7	0.89	1.33	315.2	4.0E+05	130.2	146.4	0.032	0.041
8	0.84	0.61	315.2	3.5E+05	127.3	151.4	0.029	0.041
9	0.82	0.32	315.2	3.1E+05	125.0	152.4	0.028	0.041
10	0.77	0.19	315.2	3.1E+05	136.1	177.1	0.025	0.043
11	0.74	2.08	315.2	5.1E+05	159.4	214.4	0.025	0.045
12	1.23	4.00	1386.4	1.1E+05	15.7	12.7	0.074	0.049
13	1.10	0.60	1386.4	7.0E+04	12.9	11.5	0.066	0.052
14	0.95	1.98	4154.5	3.1E+05	14.1	15.1	0.038	0.043
15	0.97	2.13	4154.5	2.9E+05	13.4	14.0	0.042	0.046
16	1.34	2.19	4154.5	4.1E+05	19.0	14.3	0.080	0.045
202	2.23	2.76	564.4	9.9E+04	40.7	18.4	0.188	0.039
203	3.20	2.79	564.4	1.5E+05	60.3	19.0	0.385	0.038
204	3.99	2.62	564.4	1.8E+05	78.7	18.8	0.671	0.038
205	4.59	2.62	564.4	2.1E+05	90.4	18.8	0.886	0.038
207	2.18	1.31	564.4	9.2E+04	41.0	19.1	0.174	0.038
208	2.49	2.79	564.4	1.1E+05	46.2	18.8	0.231	0.038
209	0.88	3.20	564.4	4.1E+04	15.8	18.8	0.027	0.038
210	0.89	2.79	181.4	6.0E+04	30.4	36.8	0.021	0.030
211	1.42	2.49	181.4	9.8E+04	50.8	38.2	0.054	0.031
212	1.81	2.49	181.4	1.3E+05	65.2	38.2	0.089	0.031
213	2.57	2.49	181.4	1.8E+05	103.8	39.6	0.211	0.031
214	2.97	2.49	181.4	2.0E+05	111.7	36.8	0.281	0.030
215	3.35	2.49	181.4	2.4E+05	135.5	39.6	0.359	0.031
217	3.85	1.97	181.4	2.5E+05	144.7	36.8	0.471	0.030
222	0.61	2.79	181.4	3.6E+04	18.1	32.2	0.009	0.030

For each experiment, the acoustic data were collected at 1 second intervals over a one minute period every 10 minutes and continued for as long as 650 hours. In some

cases, the record had hundreds of thousands of data points. Each point measured the distance from the acoustic array to the point in the water column that was dense enough to cause the acoustic signal to be reflected back to the transponder. In most cases this point was the sediment bed, however excessive suspended sediment in the water column and misadjusted transponder sensitivity did cause spurious or multiple returns. Also, as the scour hole developed, sediment mobilized by the turbulence and horseshoe vortex within the scour hole had the potential to generate false bottom returns if the transponder sensitivity was too high. Figure 3-5 shows an example of a time series with such effects.

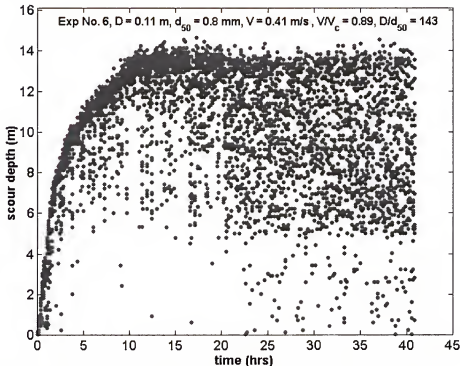


Figure 3-5. Acoustic record masked by false returns due to mobilized solids

As the scour hole depth increased, the spread of the beam caused a larger acoustic footprint at the bed. In experiments with the 0.9 meter cylinder, the scour hole became deep enough that returns were generated by a portion of the acoustic beam striking the sides of the cylinder itself. The result was multiple distinct lines of acoustic shadows that

appeared as multiple bed locations above the actual bed level (Figure 3-6).

All of these false returns had to be removed before the time series data could be studied in detail. Because of the varied nature of the false returns, no single automated procedure could be relied upon to process the data. Each data set had to be examined and processed manually before an automated filter could be applied to reduce the data set.

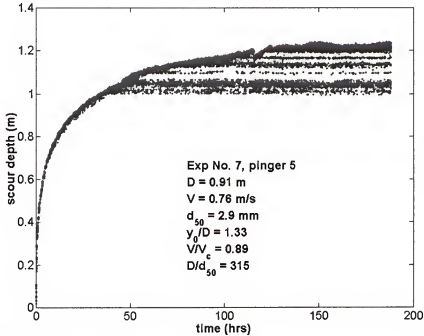


Figure 3-6. False bed shadow effects caused by acoustic returns from the side of the cylinder

The first step in the procedure was to determine the reference bed level. This level was the distance from the unscoured, compacted bed to each array. Unfortunately, the actual distance from the unscoured bed to the array mount was not available for all experiments. In these cases, initial return values generated after filling the flume and prior to initiating the flow were used and where possible these values were cross referenced and verified against the video data.

Once the data set was corrected to the reference level, returns occurring above this level (i.e., returns from suspended sediment in the water column above the bed) were

removed from the record. The record was then imported to the Scatter Point Module of the SMS program (a commercial two-dimensional CFD grid development program by Brigham Young University and Environmental Modeling Systems Inc.). This program allowed the very large data sets to be examined and manipulated graphically. In this way false signals much deeper than the actual bed and returns from suspended sediment in the water column could be recognized and removed.

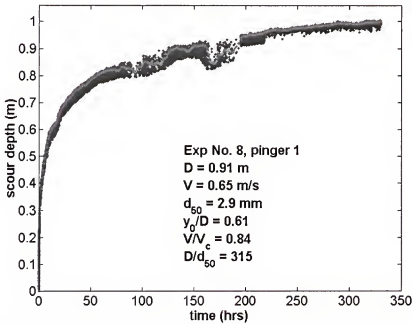


Figure 3-7. Acoustic return band and final processed scour depth time series envelop

The resulting data set consisted of a band of acoustic returns about the actual bed level. The final step was to filter this band of returns to obtain an envelope line tracing the upper edge of the band. The data set was divided into bins by time. The maximum scour depth within each bin was determined and that point was reassigned a time mark corresponding to the center of the bin. The resulting time series was divided into one-third of the number of bins and the three data points within each of these new bins were averaged to smooth the series. To further smooth the data set, a moving average was

taken of this final series over three neighboring points. Figure 3-7 shows the return band and the final trace result of the filtering and smoothing procedure for pinger 1 of Experiment 8.

Video Time Series Data Processing

Unlike the output of the acoustic arrays, the video data required time consuming manual post-test data reduction. Hundreds of hours of video tape were watched and the scour hole depth was recorded at regular intervals from the length scales attached to the inside of the test cylinders. Data reduction consisted of recording the location of the unscoured bed as measured on the length scale and the time.

The video procedure allowed direct examination of the test conditions and provided a verified reference bed level. Also, the video tape served as a permanent record for later review and evaluation of anomalies. One such anomaly was any decrease in scour depth (i.e., filling of the scour hole). Review of the video record showed such events were due to sediment sliding down the sides of the hole. It was not possible to determine the cause of such anomalies from the acoustic data.

Unfortunately, during several tests, the video camera tracking mechanism did not maintain the internal cameras at the level of the bottom of the scour hole. When this occurred the water-sediment interface line at the bottom of the hole could not be seen on the video tape and gaps were left in the time series record. Failures of the lighting system also interfered with the collection of video data and resulted either in gaps in the record or made the video record unusable.

Comparison of the processed acoustic and video records showed that the video time series generally measured the deepest scour. For this reason and the availability of a

reviewable record, analysis was conducted preferentially on the video data. When both video and acoustic data were available, the video data usually tracked well with pinger 1 or pinger 9. Therefore, when video data was not available, the maximum of pingers 1 and 9 was used.

The arrays for pingers 1 and 9 were located on the forward quarters of the cylinder. The bed in these locations was subject to continuous flow and was forward of the flow separation point. Pinger 5 was at a disadvantage because it was located in the stagnation plane of the cylinder and the downflow had to develop before significant scour began. Generally, this resulted in a delay in the time series of the scour hole development. Since pingers 1 and 9 did not show such a delay, the pinger 5 record was not a true reflection of the time history of the maximum scour hole development. The one exception was experiment 208. In experiment 208 video data was not available and the acoustic data showed a very low scour rate at the start of the experiment. Pinger 5 minimized the low scour rate period allowing a better fit for the data.

The experiments are numbered sequentially. The tests conducted at the Conte USGS-BRD Laboratory are numbered 1 through 16. Experiment numbers for tests conducted at the Hydraulics Laboratory at the University of Auckland were prefaced by a "2" to distinguish them from the Conte Laboratory experiments. Where numbers are missing, experiment time histories do not exist or problems with the conduct of the experiments or data collection invalidated the experiments. For example, tests 218 through 221 were continuations of test 217 with successively increased water velocities. These tests did not start from an unscoured condition and the time series would not have been valid. Such tests were conducted not to collect time series data, but to determine the

equilibrium scour depth in order to expand the available data set.

Curve Fitting of the Time Series Data

Only in the live bed scour experiments could it be definitively determined that equilibrium had been reached. Since equilibrium was reached quickly in these experiments, the test could be continued to verify the final state. Nearly all of the clear water scour experiments were probably stopped prior to equilibrium. Many of these experiments continued for hundreds of hours and continuing them to equilibrium became impractical. In such cases, the data sets had to be extrapolated to determine the equilibrium scour depth. Also, it was advantageous for the time rate analysis to have a continuous time series curve from which the time rate of change of the scour depth could be calculated.

Many authors have tried to describe the temporal development of local scour depth with empirical formulas. Usually these formulas take the form of an exponential or a power curve with a single time constant (Ahmad 1953, Liu et al. 1961, Shen et al. 1966, Sumer et al. 1992, Melville and Chiew 1999). However, the use of a single time constant fails to account for the multiple phases of the scouring process as described by Melville (1975), Nakagawa and Suzuki (1975) and Ettema (1980) and verified by Gosselin (1997).

Bertoldi and Jones (1998) extrapolated short time series data sets to an equilibrium condition by fitting Eq. 3-1 to their data,

$$d_s(t) = a \left[1 - \frac{1}{(1 + abt)} \right] + c \left[1 - \frac{1}{(1 + cdt)} \right]. \quad (3-1)$$

Gosselin (1997) found that this equation provide a satisfactory approximation to his data sets. By incorporating two time constants, Eq. 3-1 can account for the transition of the scouring process from one dominated by a flat-bed sediment transport due to

accelerations caused by the distortion of the streamlines to a process dominated by the mobilization of sediment by the horseshoe vortex.

A similar equation in both form and behavior is

$$d_s(t) = a[1 - \exp(-bt)] + c[1 - \exp(-dt)]. \quad (3-2)$$

Both Eq. 3-1 and 3-2 were used in the analysis and extrapolation of the time series data reported here. The video or filtered acoustic time series data sets were imported to a commercial program called TableCurve 2D (by SPSS Science). This program performs a least-squares fit of multiple coefficient equations and provided a rapid first estimate of the coefficients (a, b, c, d) in Eq. 3-1 and 3-2.

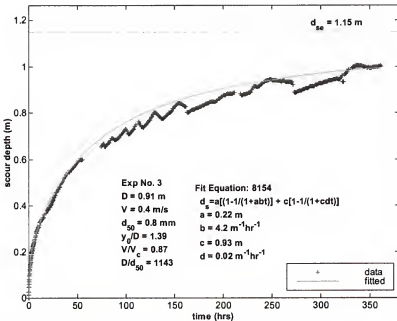


Figure 3-8. Fit curve optimization with extreme avalanching

The fitting program did not account for the significance of the rate of change of scour depth and tended to bias the fit away from early stages of the scour hole development where the depth was changing most rapidly. The TableCurve program also tended to fit a curve through the center of the data band. It is more desirable to fit a curve

to an envelope of the scour depth. Figure 3-8 shows such a case where periodic avalanching of material into the scour hole would bias TableCurve away from the optimum fit. Therefore, the result obtained from TableCurve 2D was used only as an initial fit. After fitting with TableCurve, the time series was imported into a spreadsheet program (Excel). In the spreadsheet program, the fit coefficients were adjusted to improve the least squares correlation (r^2) value while ensuring that the overall scour process was fitted.

Though similar, the behavior of Eq. 3-1 and 3-2 describe different processes. The most notable difference occurred in the early stages of scour hole development. When the flow intensity (V/V_c) rose above approximately 1.5, Eq. 3-2 did a slightly better job of describing the rapid initial scour rates. When the initial rate was slower or there existed a nearly linear stage as in very low flow intensity experiments, Eq. 3-1 gave a significantly better approximation.

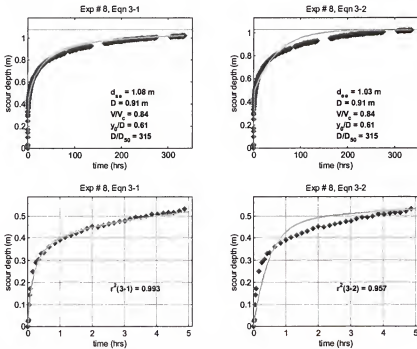


Figure 3-9. Comparison of fit equation performance for Experiment 8

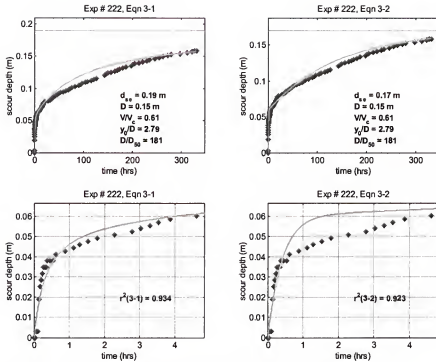


Figure 3-10. Comparison of fit equation performance for Experiment 222

Figures 3-9 and 3-10 show two low flow intensity situations. In Figure 3-9, Eq. 3-1 fits a slow approach to equilibrium best (approximately 50 to 300 hours) and is in better agreement with the data in the first 6 hours.

Figure 3-10 shows the data for even lower flow intensities (V/V_c of 0.6 verses 0.84). For this condition, the linear or flat region between 50 and 250 hours is more pronounced. Neither Equation 3-1 nor 3-2 can approximate this region well. However, Equation 3-1 fits the data better overall. In addition, Figure 3-10 shows that the Equation 3-2 fit develops a pronounced knee in the first 6 hours that is not reflected in the data.

Figure 3-11 compares the fit equations for a much higher flow intensity ($V/V_c = 3.85$). This test reached equilibrium rapidly and the test was continued to verify that equilibrium had been reached. Equation 3-2 fits the rapid rise to this known equilibrium depth better than Equation 3-1. This was true for all tests with flow intensities above approximately 1.5.

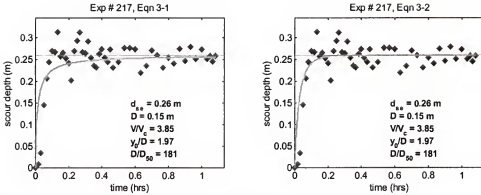


Figure 3-11. Comparison of fit equation performance for Experiment 217

Table 3-4 lists the best fit equation for each experiment and its corresponding coefficients. Column 8 (r^2) is the root mean square correlation between the data and the calculated fit equation. This value is much lower for the live bed scour experiments due to the large fluctuations in scour depth that occur as bed forms propagate through the scour hole. Column 9 (fit d_{se}) is the equilibrium scour depth determined from the fit equation (the sum of coefficients a and c). Column 10 (T_{75}) is the time at which the fit curve will reach 75% of the equilibrium scour depth. Appendix A contains the plots of these curves with the time series data sets.

Column 2 of Table 3-4 indicates whether or not equilibrium was reached during the experiment. In several of the clear water scour experiments the scouring process stopped suddenly (cut-off). This phenomena was not reported previously in the literature and was later associated with the presence of suspended fine sediment in the water column.

Storm water runoff which occurred during tests at the Conte USGS-BRD Laboratory introduced suspended fine sediments into the water column. When the fine sediment laden water reached the flume both the scour rate and the equilibrium scour depths were impacted.

Table 3-4. Fit curve parameters and computations summary

Exp. No.	Equil.	Eq.	a ^(b)	b ^(c)	c ^(b)	d ^(c)	r ²	fit d _{sc} (m)	T ₇₅ (hrs)	adj. d _{sc} (m)
1	cut off	3-1	0.05	55.0	0.13	0.3	0.985	0.18	49	0.18
2	cut off	3-1	0.17	5.0	0.24	0.05	0.997	0.41	115	0.41
3	no	3-1	0.22	4.2	0.93	0.015	0.994	1.15	161	1.15
4	no	3-1	0.3	2.0	0.65	0.01	0.997	0.95	270	0.99
5	cut off	3-1	0.1	20.0	0.33	0.16	0.999	0.43	40	0.52
6	cut off	3-1	0.08	10.0	0.1	2	0.996	0.18	9	0.23
7	no ^(a)	3-1	0.6	4.5	0.77	0.04	0.998	1.37	42	1.39
8	no ^(a)	3-1	0.48	10.0	0.6	0.045	0.997	1.08	46	1.12
9	no ^(a)	3-1	0.35	12.0	0.6	0.026	0.998	0.95	98	0.95
10	no ^(a)	3-1	0.28	14.0	0.47	0.01	0.996	0.75	321	0.75
11	no ^(a)	3-1	0.5	1.3	0.75	0.0058	0.996	1.25	326	1.33
12	yes	3-1	0.25	5.50	0.14	0.25	0.996	0.39	17	0.39
13	yes	3-1	0.12	22.0	0.168	0.5	0.996	0.29	17	0.30
14	no	3-1	0.26	0.5	0.74	0.008	0.999	1.00	343	1.00
15	cut off	3-1	0.4	0.4	0.75	0.01	0.991	1.15	227	1.15
16	yes	3-1	0.45	0.5	0.9	0.007	0.997	1.35	273	1.35
202	yes	3-2	0.14	25.0	0.075	2.5	0.969	0.22	0.16	0.22
203	yes	3-2	0.11	30.0	0.13	40.0	0.917	0.23	0.04	0.23
204	yes	3-2	0.25	45.0	0	0	0.909	0.25	0.03	0.25
205	yes	3-2	0.265	50.0	0	0	0.926	0.27	0.03	0.27
207	yes	3-2	0.11	40.0	0.07	5.5	0.970	0.18	0.09	0.18
208	yes	3-2	0.18	28.0	0.05	2.0	0.992	0.22	0.09	0.22
209	no ^a	3-1	0.075	56.0	0.075	2.3	0.996	0.15	7	0.15
210	no ^a	3-1	0.1	120.0	0.14	0.65	0.997	0.24	15	0.24
211	yes	3-1	0.12	200.0	0.05	150.0	0.620	0.17	0.19	0.17
212	yes	3-2	0.21	45.0	0.043	10.0	0.868	0.25	0.04	0.25
213	yes	3-2	0.2	60.0	0.05	4.0	0.754	0.25	0.04	0.25
214	yes	3-2	0.2	50.0	0.03	2.0	0.677	0.23	0.04	0.23
215	yes	3-2	0.17	50.0	0.08	20.0	0.766	0.25	0.04	0.25
217	yes	3-2	0.15	50.0	0.11	18.0	0.763	0.26	0.04	0.26
222	no ^a	3-1	0.06	55.0	0.13	0.07	0.990	0.19	191	0.19

^(a) Equilibrium was not reached, but the final depth was likely close to equilibrium.

^(b) units are meters.

^(c) units are (meter-hour)⁻¹ for Eq. 3-1 and hour⁻¹ for Eq. 3-2.

Figure 3-12 is an acoustic time series record that illustrates the effect of suspended fine sediment on the scour depth. At approximately 175 hours, the scouring process was halted. The video tape of this experiment reveals the effects of fine sediment directly. Though it is difficult to discern the suspended sediment, the appearance of the bed of the scour hole changes and the movement of sediment stops. Whether the appearance change was due to fine sediment settling into the hole or is an optical effect

caused by the cessation of motion of the sediment near the bed cannot be determined from the video.

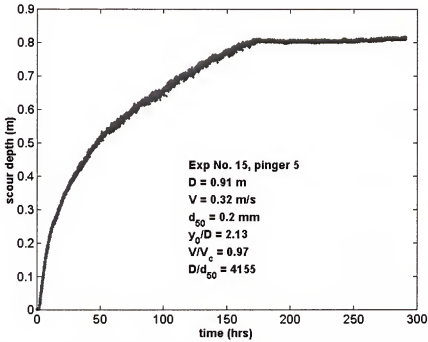


Figure 3-12. Scouring cut-off due to suspended fine sediments for Experiment 15

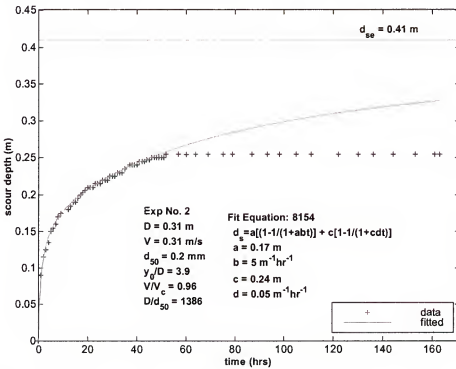


Figure 3-13. Curve fitting accounting for scouring cut-off for Experiment 2

Sheppard (2002a, 2002b) and Smyre (2002) investigated this phenomenon and noted a theory explaining the effect. "The presence of the suspended fine sediment causes a thickening of the viscous layer near the boundary and a reduction in the turbulence intensity (and therefore a reduction in Reynolds stresses) in the water column. It is therefore quite possible that the reduced scour depths are due to a reduction in bed shear stress caused by the suspended fine sediment" (Sheppard 2002a, p. 199).

For this study, the effects had to be accounted for in the fit curve to obtain useful information for the affected experiments. Figure 3-13 is an example of a fit curve adjusted to compensate for the sudden scouring cut-off associated with suspended fine sediment. In this experiment, the cut-off phenomenon was estimated to have begun at approximately 40 hours. The time series prior to this was heavily weighted and the illustrated curve was fitted to the data set. In the case of a cut-off data set, no attempt is made to adjust the equilibrium scour depth to the maximum scour depth obtained by point gage.

3.4 Variation of the Time Scale

Using the fitted curve, a time to reach some percentage of equilibrium could be calculated. This time scale (T_{xx}) can be used to exam the long term behavior of the scouring process. For this study the commonly examined time to reach 75% of the equilibrium scour depth (T_{75}) was chosen as the time scale. Table 3-4 lists this value for each experiment. To better examine the effects of the flow and sediment conditions on the time scale, the experiments were grouped by constant V/V_c , y_0/D and D/d_{50} . Table 3-5 lists these experiments and their grouping parameters. The groups were arranged to maximize the range covered by the variable parameter and minimize the range for the

other two. Ideally, the common parameters are exactly the same for all tests within the group; however that was not possible with this data set.

Table 3-5. Time scale study experiment groups

Experiments	Variable Parameter	V/V _c Range	D/d ₅₀ Range	y ₀ /D Range
210, 211, 213, 214, 215, 222	V/V _c	0.6-3.4	181	2.5-2.8
202, 203, 204, 205, 208, 209	V/V _c	0.9-4.6	564	2.6-3.2
2, 12	V/V _c	0.96-1.23	1386	3.9-4.0
14, 16	V/V _c	0.95-1.34	4155	2-2.2
3, 7	D/d ₅₀	0.9	315-1143	1.3-1.4
16, 211	D/d ₅₀	1.3-1.4	181-4155	2.2-2.5
208-213	D/d ₅₀	2.5-2.6	181-564	2.5-2.8
203-215	D/d ₅₀	3.2-3.4	181-564	2.5-2.8
10, 11	y ₀ /D	0.7-0.8	315	0.2-2.1
3, 4	y ₀ /D	0.9	1143	1-1.4
5, 8, 9	y ₀ /D	0.8	315-381	0.3-4.2
12, 13	y ₀ /D	1.1-1.2	1386	0.6-4
6, 210	y ₀ /D	0.9	143-181	2.8-11
1, 209	y ₀ /D	0.9	518-564	3.2-10.4
202, 207	y ₀ /D	2.2	564	1.3-2.8

Flow Intensity Effect

Figure 3-14 shows the effect of flow intensity (V/V_c) on the time scale. All plots show the time scale is decreasing with increasing flow intensity. However, the decreasing T₇₅ trend for increasing V/V_c in the clear water scour range is based only on the two clear water scour points in Figure 3-14 plot (a). All other plots contain only one clear water scour data point and so reflect a change from clear water scour to live bed scour conditions. In the live bed range, there is a solid trend of decreasing T₇₅ with increasing flow intensity seen in plots (a) and (b). Crossing from clear water to live bed scour also produces a downward trend.

Two flow intensity effects control the time scale. Increasing the flow intensity increases the rate at which material is removed from the scour hole. This effect tends to reduce the time scale. On the other hand, equilibrium scour depth increases (in general)

with increasing flow intensity. This effect tends to increase the time to equilibrium.

Figure 3-14 indicates that the latter effect is dominant and the time to equilibrium tends to decrease with increasing flow intensity.

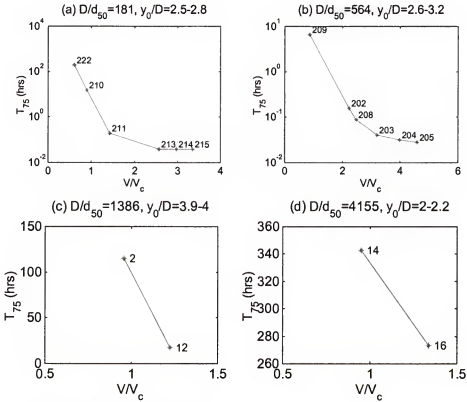


Figure 3-14. Variation of T_{75} with V/V_c

Using T_{90} as his time scale, Baker (1978) found the same decrease in time scale with increasing flow intensity in the live bed scour range; however he found that increasing flow intensities in the clear water scour range gave higher values of T_{90} . Baker's use of surface drogues to measure velocity limited the accuracy of his velocity values and thus his flow intensity ranges are questionable.

Using a more extensive data set, Melville and Chiew (1999) supported Baker's conclusion that increasing the flow intensities in the clear water scour range tended to increase the time scale, thus indicating that the increasing equilibrium depth effect was

dominant. They used a more complex time scale (t_e), defining it as the time at which the scour hole reached a depth where the rate of increase in depth does not exceed five percent of the pier diameter in the succeeding 24 hour period. Figure 3-15 shows plots of Melville and Chiew's data for t_e versus V/V_c grouped by D/d_{50} and y_0/D .

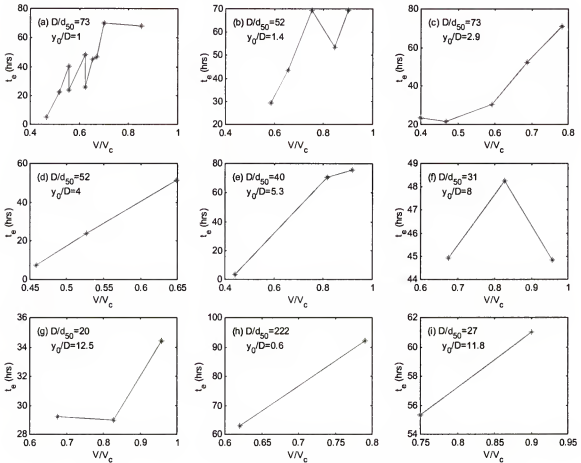


Figure 3-15. Time to equilibrium (t_e) vs. V/V_c from Melville and Chiew (1999)

In Figure 3-15, there is a general trend of increasing equilibrium time with increasing flow intensity for the clear water scour range. This trend is most pronounced in the 0.4 to 0.6 range in plots (a) through (e) and (h). In the 0.7 to 1 range, plots (b), (e), (f) and possibly (a) and (i) show that t_e is relatively independent of flow intensity. In addition, plot (a) shows two pairs of experiments with the same values of V/V_c as well as

D/d_{50} , and y_0/D but with very different equilibrium times, indicating a problem with the data or the choice of a time scale.

From the University of Florida data reported here and that reported by Baker (1978), it can be concluded that increasing flow intensity decreases the time to equilibrium in the live bed scour range. In this range the increased rate of sediment removal due to increased V/V_c dominates. The trend is less clear in the clear water scour range. There is strong evidence that increasing flow intensity increases the time to equilibrium and the effect of increasing equilibrium scour depth with increasing V/V_c dominates. However, inconsistencies in the data make this trend less certain.

It has been noted by researchers studying equilibrium local scour depth (d_{se}) that the variation of d_{se} with flow intensity is not linear in the clear water scour range. In fact, the rate of change of d_{se} with V/V_c becomes more gradual as V/V_c approaches 1. It may be that for a range of roughly $0.4 < V/V_c < 0.7$, the increase in d_{se} with V/V_c is rapid enough to dominate the increase in sediment removal caused by increasing V/V_c . This would increase the time scale as V/V_c increased. As V/V_c increases above approximately 0.7 to 0.8 toward 1, the rate of change of d_{se} slows. Thus, the opposing effects on the time scale may balance one another and the time to equilibrium becomes relatively independent of V/V_c . Once V/V_c is greater than 1, the rate of sediment removal becomes dominant and the time scale decreases with increasing V/V_c .

Normalized Sediment Size Effect

Figure 3-16 shows the University of Florida T_{75} data plotted against the normalized sediment size (D/d_{50}) and grouped by nearly constant V/V_c and y_0/D . The plots show a decided trend of increasing time scale with increasing D/d_{50} .

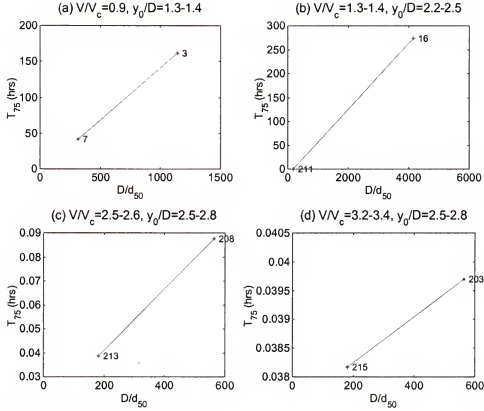


Figure 3-16. Variation of T_{75} with D/d_{50}

Figure 3-17 shows time scale plots for this grouping of the Melville and Chiew (1999) data. This data shows the same trend, increasing time scale with increasing D/d_{50} . The Melville and Chiew data is limited to $V/V_c = 0.9$ but covers a range of y_0/D from 0.5 to 6. The University of Florida data is limited in y_0/D (1.3 to 2.8) but covers a nice range of V/V_c from 0.9 to 3.4. Unfortunately, neither of the data sets contains experiments which may be grouped with V/V_c less than 0.9.

Despite the lack of information below V/V_c of 0.9, the data strongly indicates that the time to equilibrium increases for larger D/d_{50} . To explain this trend, the effect of D/d_{50} on the equilibrium scour depth should be considered. Most researchers agree that the scour hole size increases as the cylinder diameter increases. Increasing the size of the scour hole requires that a larger volume of sediment must be removed. Since the volume

of sediment increases as D^3 and the area over which the sediment is mobilized and transport increases as D^2 , it should take a longer time for equilibrium to be reached as D/d_{50} increases, as indicated in Figures 3-16 and 3-17.

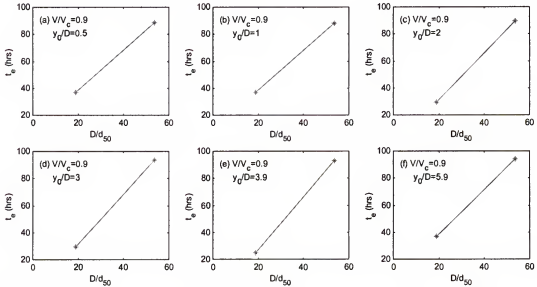


Figure 3-17. Time to equilibrium (t_e) vs. D/d_{50} from Melville and Chiew (1999)

On the other hand, since a reduction in d_{50} leads to a reduction in the critical shear stress, reducing the sediments size might be expected to increase the sediment transport and reduce the time to equilibrium for increasing D/d_{50} . However, to maintain the flow intensity (V/V_c) constant, the shear stress on the bed must be reduced as the critical shear stress is reduced (i.e., as V_c decreases, V must decrease). Thus, a reduction in d_{50} must be accompanied by a reduction in actual flow velocity and the sediment transport is again reduced, leading to an increase in time scale with increasing D/d_{50} .

Flow Aspect Ratio Effect

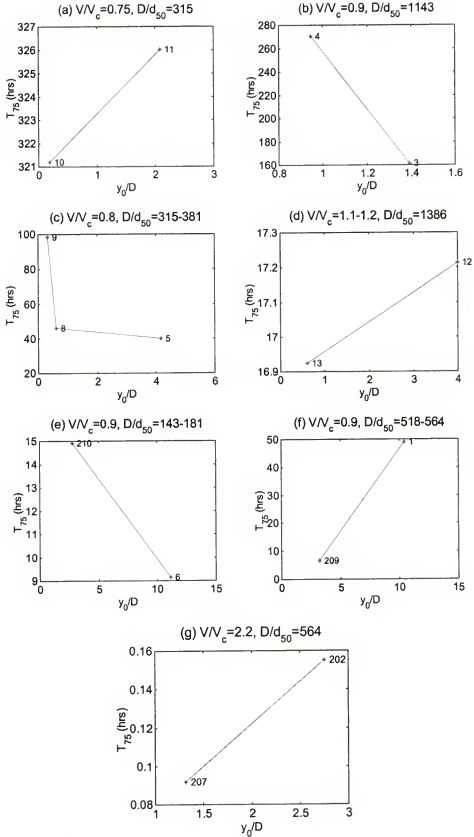
Figure 3-18 shows the University of Florida T_{75} data plotted against the flow aspect ratio (y_0/D) and grouped by nearly constant V/V_c and D/d_{50} . Figure 3-19 shows the same plots of t_e from Melville and Chiew (1999). Neither set of plots show a clear

trend for time scale as a function of y_0/D . Plots (a) and (d) of Figure 3-18 show little change in T_{75} with y_0/D and plots (b), (c) and (e) show marked increases in T_{75} with increasing y_0/D . However, plots (f) and (g) show the opposite effect and plot (c) shows no change for y_0/D between approximately 1 and 4. The plots in Figure 3-19 are equally inconsistent.

Melville and Chiew reported an increase in their nondimensional time scale (Vt_e/D) with increasing y_0/D . When nondimensionalizing in this way, decreasing the pier size (D) to increase y_0/D would tend to increase the nondimensional time scale if t_e were constant.

If Figure 3-17 from the previous section is taken as a series the plots, there is a progression from y_0/D of 0.5 to 5.9. The t_e values are remarkably consistent between plots, suggesting that the time scale may be independent of the flow aspect ratio (y_0/D).

Ultimately, no real conclusions can be made on the variation of the time scale with flow aspect ratio from the data sets available.

Figure 3-18. Variation of T_{75} with y_0/D

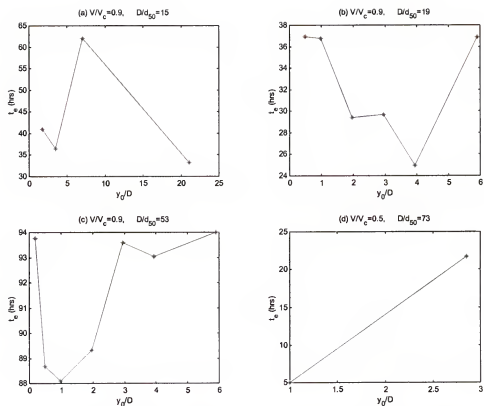


Figure 3-19. Time to equilibrium (t_e) vs. y_0/D from Melville and Chiew (1999)

CHAPTER 4

PHYSICS-BASED MODEL FOR THE DEVELOPMENT OF A SCOUR HOLE WITH TIME

4.1 Modeling the Scour Hole as a Function of Time

Approximating the Scour Hole

It is desirable to be able to model the scour hole development as a function of time based on the physics of the scouring process. The model must be simple enough to be used for engineering design. Unknowns in the sediment transport process and the requirement for simplicity necessitate empirical approximations, but these should be minimized and restricted to individual phenomena (e.g., sediment transport). This will allow for a more rational extrapolation of our understanding of the scour process beyond the range of conditions for which experimental data exists.

As noted in Chapter 2, 3-D numerical simulations of the scour process do not currently provide a practical method for determining the development of the scour hole as a function of time on the prototype scale. The model presented here will assume that the geometry of the scour hole can be approximated as an inverted frustum of a right circular cone, that the scour hole remains symmetric with time and that the sides of the hole maintain a slope equal to the submerged angle of repose of the sediment.

Figure 4-1 shows a series of post-test contour plots of the scour holes for several of the clear water scour experiments presented in this dissertation. These plots, and Figure 2-10 from Gosselin (1997), show that the local scour hole around a circular cylinder is highly symmetrical.

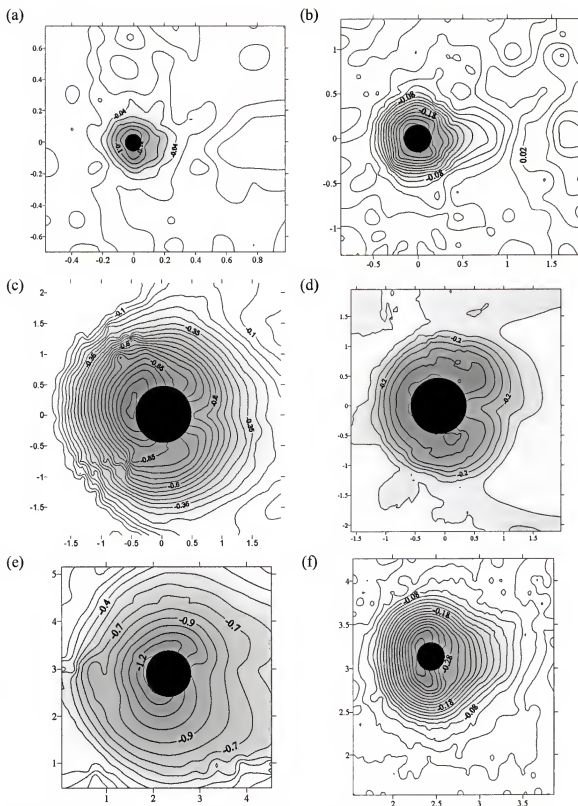


Figure 4-1. Contour plots of (a) Exp. 1, $D = 0.1$ m; (b) Exp. 2, $D = 0.3$ m; (c) Exp. 3, $D = 0.9$ m; (d) Exp. 4, $D = 0.9$ m; (e) Exp. 7, $D = 0.9$ m; (f) Exp. 12, $D = 0.3$ m (Sheppard 2002b)

In accordance with the observations of Melville (1975), Ettema (1980), and Nakagawa and Suzuki (1975), the erosion in the scour hole is assumed to be limited to a region immediately adjacent to the cylinder. This area (the erosion zone) is fed by sediment avalanching (sliding) down the sides of the scour hole. This maintains the sides of the scour hole at a constant angle. Observations by Shen et al. (1966), Melville (1975) and Gosselin (1997) of low shear stress areas on the side of the scour hole and high shear stress areas at the base of the cylinder (discussed in Chapter 2) directly support this assumption. Using the observations of Nakagawa and Suzuki (1975), the width of the erosion zone is assumed to be constant with scour hole depth and is only a function of the diameter of the cylinder.

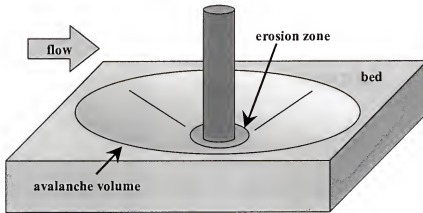


Figure 4-2. Conceptual sketch of ideal scour hole

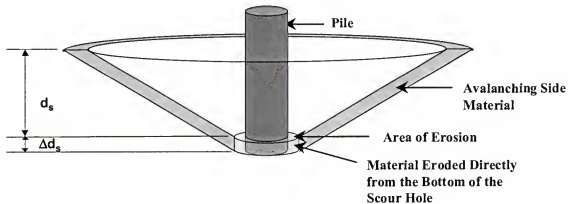


Figure 4-3. 3D definition sketch of ideal scour hole

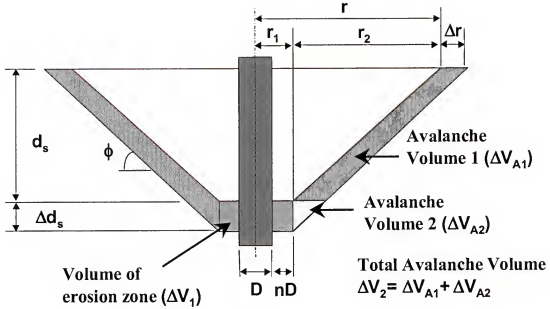


Figure 4-4. Definition sketch of scour hole

Figure 4-2 is a three dimensional conceptual sketch of an idealized scour hole.

Figures 4-3 defines the scour hole sediment volumes, and Figure 4-4 is a detailed definition sketch.

Derivation of the Volume Rate of Change of the Scour Hole

The following relationships can be developed using the variables defined in

Figures 4-3 and 4-4:

$$(a) \ r_1 = (n + \frac{1}{2})D, \ (b) \ r_2 = \frac{d_s}{\tan \phi}, \ \text{and} \ (c) \ \Delta r = \frac{\Delta d_s}{\tan \phi}. \quad (4-1)$$

From Figure 4-4, the volume of the annulus at the base of the scour hole (ΔV_1) is

$$\Delta V_1 = \Delta d_s \left[\pi r_1^2 - \pi \left(\frac{1}{2} D \right)^2 \right]. \quad (4-2)$$

Combining Eq. 4-1(a) and 4-2 gives

$$\begin{aligned} \Delta V_1 &= \Delta d_s \pi \left[\left(n + \frac{1}{2} \right)^2 D^2 - \frac{1}{4} D^2 \right] = \Delta d_s \pi D^2 (n^2 + n) \\ \text{or } \Delta V_1 &= \Delta d_s \pi D^2 n(n+1). \end{aligned} \quad (4-3)$$

The volume of avalanche material above the bottom of the scour hole (i.e., ΔV_{A1} in Figure 4-4) is

$$\Delta V_{A1} = \int_0^{d_s} [\pi(r + \Delta r)^2 - \pi r^2] dy = \int_0^{d_s} [2\pi r \Delta r + \pi \Delta r^2] dy. \quad (4-4)$$

Combining Eq. 4-1 and 4-4 gives

$$\Delta V_{A1} = \int_0^{d_s} \left[2\pi \left[\frac{1}{2} D(2n+1) + \frac{d_s}{\tan \phi} \right] \frac{\Delta d_s}{\tan \phi} + \pi \left(\frac{\Delta d_s}{\tan \phi} \right)^2 \right] dy. \quad (4-5)$$

Assuming that Δd_s is not a function of d_s , Eq. 4-5 can be rearranged and integrated to yield

$$\Delta V_{A1} = \pi \frac{\Delta d_s}{\tan \phi} \left[D(2n+1) + \frac{\Delta d_s}{\tan \phi} \right] \int_0^{d_s} dy + 2\pi \frac{\Delta d_s}{\tan \phi} \int_0^{d_s} \left(\frac{d_s}{\tan \phi} \right) dy.$$

Integrating gives

$$\Delta V_{A1} = \pi \frac{\Delta d_s}{\tan \phi} \left\{ \left[D(2n+1) + \frac{\Delta d_s}{\tan \phi} \right] d_s + \frac{d_s^2}{\tan \phi} \right\}. \quad (4-6)$$

Avalanche volume 2 (ΔV_{A2} in Figure 4-4) is found by subtracting the cylinder with height Δd_s and radius r_1 from the frustum of the cone with height Δd_s , base radius r_1 and upper radius $r_1 + \Delta r$. The volume of the cone is

$$V_{\text{cone}} = \frac{1}{3} \pi \Delta d_s [r_1^2 + r_1(r_1 + \Delta r) + (r_1 + \Delta r)^2] = \frac{1}{3} \pi \Delta d_s (3r_1^2 + 3r_1 \Delta r + \Delta r^2). \quad (4-7)$$

Combining Eq. 4-1 and 4-7 results in

$$\begin{aligned} V_{\text{cone}} &= \pi \Delta d_s \left[\left(\frac{D}{2} + nD \right)^2 + \left(\frac{D}{2} + nD \right) \frac{\Delta d_s}{\tan \phi} + \frac{1}{3} \left(\frac{\Delta d_s}{\tan \phi} \right)^2 \right] \\ &= \pi \Delta d_s \left[\frac{1}{4} D^2 (2n+1)^2 + \frac{1}{2} D(2n+1) \frac{\Delta d_s}{\tan \phi} + \left(\frac{\Delta d_s}{\tan \phi} \right)^2 \right] \text{ or} \end{aligned}$$

$$V_{\text{cone}} = \frac{1}{4} \pi D^2 (2n+1)^2 \Delta d_s + \frac{1}{2} \pi D (2n+1) \frac{\Delta d_s^2}{\tan \phi} + \frac{1}{3} \pi \frac{\Delta d_s^3}{\tan^2 \phi}. \quad (4-8)$$

The volume of the cylinder is

$$V_{\text{cylinder}} = \pi \Delta d_s \left(\frac{D}{2} + nD \right)^2 = \frac{1}{4} \pi D^2 (2n+1)^2 \Delta d_s. \quad (4-9)$$

Subtracting Eq. 4-9 from Eq. 4-8 gives

$$\Delta V_{A2} = V_{\text{cone}} - V_{\text{cylinder}} = \frac{1}{2} \pi D (2n+1) \frac{\Delta d_s^2}{\tan \phi} + \frac{1}{3} \pi \frac{\Delta d_s^3}{\tan^2 \phi}. \quad (4-10)$$

The total avalanche volume (ΔV_2) becomes

$$\begin{aligned} \Delta V_2 &= \Delta V_{A1} + \Delta V_{A2} \\ &= \pi \Delta d_s \left[D(2n+1) \frac{d_s}{\tan \phi} + \left(\frac{d_s}{\tan \phi} \right)^2 \right] + \pi \left(\frac{\Delta d_s}{\tan \phi} \right)^2 d_s \\ &\quad + \frac{1}{2} \pi D (2n+1) \frac{\Delta d_s^2}{\tan \phi} + \frac{1}{3} \pi \frac{\Delta d_s^3}{\tan^2 \phi} \end{aligned} \quad (4-11)$$

Adding Eq. 4-3 and 4-11 and grouping by powers of Δd_s gives the incremental change in volume for the scour hole,

$$\begin{aligned} \Delta V &= \pi \left[nD^2(1+n) + D(2n+1) \frac{d_s}{\tan \phi} + \left(\frac{d_s}{\tan \phi} \right)^2 \right] \Delta d_s \\ &\quad + \pi \left[\frac{1}{2} D(2n+1) + \frac{d_s}{\tan \phi} \right] \frac{\Delta d_s^2}{\tan \phi} + \frac{1}{3} \pi \frac{\Delta d_s^3}{\tan^2 \phi} \end{aligned} \quad (4-12)$$

Since Δd_s is small, terms involving Δd_s^2 and Δd_s^3 may be neglected and Eq. 4-12 simplifies to

$$\Delta V = \pi \left[nD^2(1+n) + D(2n+1) \frac{d_s}{\tan \phi} + \left(\frac{d_s}{\tan \phi} \right)^2 \right] \Delta d_s. \quad (4-13)$$

The rate of change of volume of the scour hole over a given time increment (Δt) is

$$\frac{\Delta V}{\Delta t} = \pi \frac{\Delta d_s}{\Delta t} \left[nD^2(1+n) + D(2n+1) \frac{d_s}{\tan \phi} + \left(\frac{d_s}{\tan \phi} \right)^2 \right]. \quad (4-14)$$

In the limit as Δt goes to zero, Eq. 4-14 becomes

$$\frac{dV}{dt} = \pi \frac{d(d_s)}{dt} \left[nD^2(1+n) + D(2n+1) \frac{d_s}{\tan \phi} + \left(\frac{d_s}{\tan \phi} \right)^2 \right]. \quad (4-15)$$

The volume of sediment (V_s) removed is related to the volume of the scour hole (V) by $V = V_s / (1-p)$, where p is the porosity, $p = V_w / (V_s + V_w) = V_w / V$. In terms of the sediment volume, Eq. 4-15 becomes

$$\frac{dV_s}{dt} = \pi(1-p) \frac{d(d_s)}{dt} \left[nD^2(1+n) + D(2n+1) \frac{d_s}{\tan \phi} + \left(\frac{d_s}{\tan \phi} \right)^2 \right]. \quad (4-16)$$

The volume rate of transport of sediment can be described by

$$\frac{dV_s}{dt} = Q_{out} = q_{out} w \quad (4-17)$$

where Q_{out} is the rate of transport of material by volume (m^3/s) out of the scour hole, q_{out} is the rate of transport of material by volume per unit width ($m^3/s/m$ or m^2/s) and w is the width of the area over which the sediment transport function acts.

Figure 4-5 illustrates the transport of sediment out of the erosion zone around the cylinder. Sediment is mobilized in the erosion zone, lifted into the water column and transported away from the cylinder. Once the suspended sediment leaves the erosion zone, the horseshoe vortex and the background flow sweeps it downstream and out of the scour hole.

From Figure 4-5 it can be seen that the width through which the sediment passes (the width to be used in Eq. 4-17) is the circumference of the erosion zone up to the point

of separation of the flow or $w = 2(\frac{1}{2}D + nD) \beta_e = D(2n+1) \beta_e$. Downstream of this point the wake vortices will continue to mobilize the sediment and that sediment will be swept downstream. However, lacking sufficient information about the wake vortices and their transport capacity, this effect will be absorbed into the function for the transport out of the erosion zone.

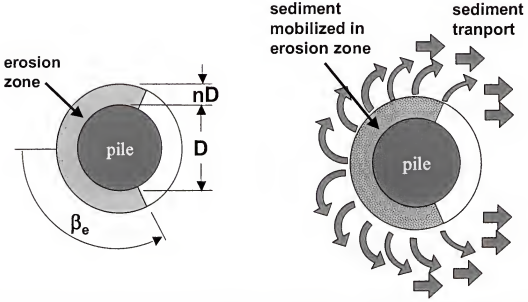


Figure 4-5. Erosion zone definition diagram and sediment mobilization and transport out of the erosion zone

Incorporating the width (w), Eq. 4-17 becomes

$$\frac{dV_s}{dt} = q_{out} D(2n+1) \beta_e. \quad (4-18)$$

Combining Eq. 4-16 and 4-18 gives the relationship between the rate of change of the depth of the scour hole and the sediment transport,

$$\pi(1-p) \frac{d(d_s)}{dt} \left[nD^2(1+n) + D(2n+1) \frac{d_s}{\tan \phi} + \left(\frac{d_s}{\tan \phi} \right)^2 \right] = q_{out} D(2n+1) \beta_e. \quad (4-19)$$

Solving for $d(d_s)/dt$ gives

$$\frac{d(d_s)}{dt} = q \frac{D(2n+1)\beta_e}{\pi(1-p)} \left[nD^2(1+n) + D(2n+1) \frac{d_s}{\tan \phi} + \left(\frac{d_s}{\tan \phi} \right)^2 \right]^{-1} \quad (4-20)$$

Defining the nondimensional scour depth as $y = d_s/d_{sc}$, Eq. 4-20 can be nondimensionalized with respect to length to give

$$\frac{dy}{dt} = q \frac{D(2n+1)\beta_e}{\pi(1-p)d_{sc}} \left[nD^2(1+n) + D(2n+1) \frac{d_{sc}y}{\tan \phi} + \left(\frac{d_{sc}y}{\tan \phi} \right)^2 \right]^{-1} \quad (4-21)$$

or

$$\frac{dy}{dt} = \frac{K_D q_{out}}{K(y)} \quad (4-22)$$

where $K_D = D(2n+1)\beta_e$ and $K(y) = \pi(1-p)d_{sc} \left[nD^2(1+n) + D(2n+1) \frac{d_{sc}y}{\tan \phi} + \left(\frac{d_{sc}y}{\tan \phi} \right)^2 \right]$.

Note that dy/dt has dimensions of inverse time and $K(y)$ has dimensions of length squared.

Eq. 4-22 assumes that no sediment will be introduced to the scour hole from upstream. To generalize the equation, an input sediment transport term is added to Eq. 4-19, which gives

$$\pi(1-p) \frac{d(d_s)}{dt} \left[nD^2(1+n) + D(2n+1) \frac{d_s}{\tan \phi} + \left(\frac{d_s}{\tan \phi} \right)^2 \right] = Q_{out} - Q_{in} \quad (4-23)$$

Expanding the transport terms, nondimensionalizing with respect to length, and simplifying by using K_D and $K(y)$ gives

$$\frac{d(y)}{dt} K(y) = K_D q_{out} - q_{in} w \quad (4-24)$$

To determine the width over which the input transport function acts, it is assumed that all bed-load material passing over the scour hole will fall into the scour hole and add

to the avalanche material. Since the turbulence around the cylinder will be greater than that upstream, it is assumed that suspended material passing over the scour hole will remain in suspension. This limits the input transport to a bed-load function. Figure 4-6 depicts this situation graphically.

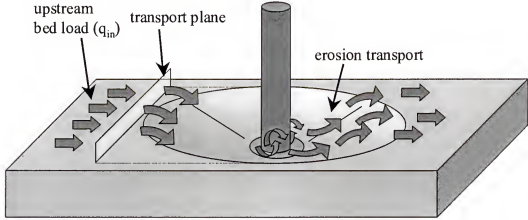


Figure 4-6. Scour sediment transport with upstream input transport

From Figure 4-6, the width (w) in Eq. 4-24 is the width of the scour hole. This width starts as the total width of the cylinder plus the erosion zone ($D + 2nD$) and grows as $2d_s/\tan\phi$. In nondimensional form this is $2y/\tan\phi$. Substituting the width (w) into Eq. 4-24 gives

$$\frac{d(y)}{dt} K(y) = K_D q_{out} - q_{in} \left[D(1 + 2n) + 2 \frac{d_{se}}{\tan\phi} y \right]. \quad (4-25)$$

Defining $L(y) = L_0 + L_1 y$, where $L_0 = D(1 + 2n)$ and $L_1 = 2d_{se}/\tan\phi$, and solving for dy/dt , Eq. 4-25 becomes

$$\frac{dy}{dt} = \frac{K_D q_{out} - L(y) q_{in}}{K(y)}. \quad (4-26)$$

4.2 Sediment Transport Functions

Eq. 4-26 is the basic model for scour hole development. To calculate the scour depth as a function of time, the sediment transport functions (q_{out} and q_{in}) must be known.

The input sediment transport is determined by the upstream conditions and is limited to bed-load, as noted previously. The output or erosion transport must be a function of scour hole depth. A number of choices for the form of the bed-load sediment transport function exist in the literature. They have been developed for flat-bed transport. All of these functions relate the transport rate to bed shear stress.

Four transport formulas have been examined here. They are Meyer-Peter Mueller (1948), Englund-Hansen (1972) modified by Neilsen (1992) for bed-load transport, Van Rijn (1993) and Einstein's bed-load formula (Einstein 1950). Table 4-1 summarizes these formulas in terms of nondimensionalized bed shear stress ($\theta = \tau_b / [\rho g (sg - 1) d_{50}]$) and critical shear stress ($\theta_c = \tau_c / [\rho g (sg - 1) d_{50}]$). Van Rijn defined $T = (\theta - \theta_c) / \theta_c$ and the nondimensional sediment size as $d_* = d_{50} [(s - 1)g / \nu^2]^{1/3}$. Einstein defined the probability of erosion (P) and determined that it followed a normal error distribution.

Table 4-1. Sediment transport formulas

Source	$\theta - \theta_c \leq 0$	$\theta - \theta_c > 0$
Meyer-Peter Mueller (1948)	0	$q = 8\sqrt{(s-1)gd_{50}^3} (\theta - \theta_c)^{1.5}$
Englund-Hansen-Neilsen (1992)	0	$q = 12\sqrt{(s-1)gd_{50}^3} \sqrt{\theta(\theta - \theta_c)}$
Van Rijn (1993)	0	$T < 3, q = 0.053\sqrt{(s-1)gd_{50}^3} d_*^{-0.3} T^{2.1}$
		$T \geq 3, q = 0.1\sqrt{(s-1)gd_{50}^3} d_*^{-0.3} T^{1.5}$
Einstein (1950)	$q = \sqrt{(s-1)gd_{50}^3} \frac{1}{27} \frac{P}{1-P}$, for all θ , where $P = 1 - \frac{1}{2} [\text{erf}(0.156/\theta - 2) + \text{erf}(0.156/\theta + 2)]$	

Figure 4-7 compares the values of the nondimensional upstream bed-load sediment transport ($q_b^* = q / \sqrt{(s-1)gd_{50}^3}$) computed by the relationships in Table 4-1.

There is no significant difference between the results for the range of live bed

experimental conditions presented here. This suggests that any of the transport equations are adequate for the model.

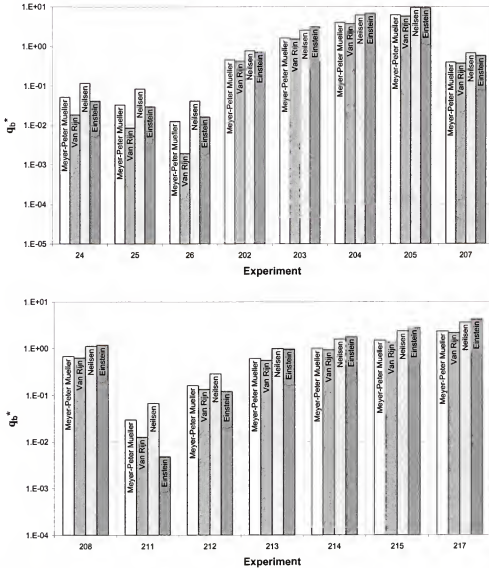


Figure 4-7. Comparison plot of upstream nondimensional bed-load sediment transport ($q_b^* = q / \sqrt{(s-1)gd_{50}^3}$) by experiment.

4.3 Modeling the Sediment Transport and Effective Shear Stress

Effective Shear Stress

The formulas of Table 4-1 have a common form with the transport (q) equal to a constant (C) times some function of bed shear stress (θ); $q = Cf(\theta)$.

It is assumed that the output sediment transport of the scour model (Eq. 4-26) can be represented by an expression of the same form as those for transport on a flat bed. Eq. 4-26 can be rewritten as

$$\frac{dy}{dt} = \frac{K_D C f_o(\theta') - L(y) q_{in}}{K(y)} \quad (4-27)$$

where C is the transport coefficient and $f_o(\theta')$ is the sediment transport formula applied to the effective shear stress (θ') on the erosion zone and q_{in} is a flat-bed sediment transport from Table 4-1 calculated from the upstream conditions.

The effective shear stress (θ') is not necessarily a measurable quantity. The actual shear stress in the erosion zone is highly episodic due to the unsteady nature of the horseshoe vortex. Also, the flow has a strong downward component. Any formula developed for flow over a flat bed can only approximate a time mean transport in the scour hole. The effective shear stress is that shear stress, averaged over the erosion zone and over time, which is necessary to provide the output sediment transport required using the given flat-bed transport formula.

A necessary assumption to complete the model is that the transport constant (C) remains constant with the depth of the scour hole. This allows the transport constant to be calculated from the start-up or initial rate of change of scour depth. The initial rate can be found from the fit functions (Eq. 3-1 and 3-2, respectively):

$$\left. \frac{dy}{dt} \right|_0 = \frac{a^2 b + c^2 d}{a + c} \quad \text{and} \quad \left. \frac{dy}{dt} \right|_0 = \frac{ab + cd}{a + c}.$$

For scour to be initiated at the structure, the local bed shear stress must exceed the critical shear stress for the sediment. It has been observed that a lower limit exists for V/V_c below which scour does not occur. Generally, this limit is placed in the flow

intensity range $0.4 \leq V/V_c \leq 0.5$. Therefore, the upstream depth averaged velocity must reach some finite value in order for the velocity near the cylinder to be sufficient for sediment transported to be initiated. Assuming that the bed shear stress (θ) is proportional to the depth averaged velocity squared,

$$\frac{\theta'_0}{\theta_u} = \frac{V_c^2}{(\epsilon V_c)} = \frac{1}{\epsilon^2}$$

where ϵ is the minimum value of V/V_c required for scour to initiate. Using $\epsilon = 0.47$ from Sheppard (2002b), the initial effective shear stress (θ'_0) can be estimated at $\theta'_0 = 4.5\theta_u$. Sumer and Fredsoe (2002) measured bed shear stress under a horseshoe vortex and normalized it with the upstream bed shear stress. The normalized shear stress reaches a peak of approximately 5, indicating that the above initial shear stress calculation agrees well with experimental data.

Solving Eq. 4-27 for the transport constant gives

$$C = \left[\frac{dy}{dt} \Big|_0 K_0 + L_0 q_{in} \right] \left[K_D f_o(\theta'_0) \right]^{-1} \quad (4-28)$$

where K_0 and L_0 are $K(y)$ and $L(y)$ at $y = 0$, respectively, and $K_0 = \pi(1-p)d_{sc}nD^2(1+n)$.

Similarly, Eq. 4-27 can be rearranged to give the transport function,

$$f_o(\theta') = \left[\frac{dy}{dt} K(y) + L(y) q_{in} \right] (K_D C)^{-1}. \quad (4-29)$$

Effective Shear Stress as a function of Scour Depth

Each transport function in Table 4-1 was used to obtain q_{in} for the experiments in Table 3-1. With the same transport function, Eq. 4-30 was solved for the effective shear stress as a function of normalized scour depth for each experiment listed in Table 3-1.

Figures 4-9 to 4-17 show the shear stress profiles for a number of experiments.

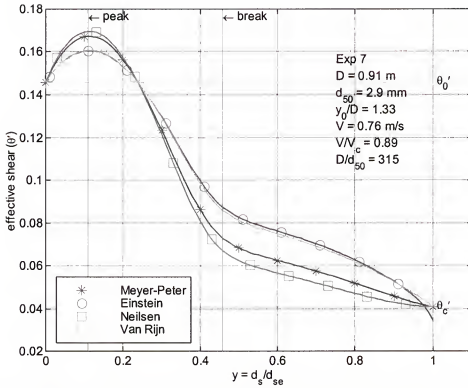


Figure 4-8. Effective shear stress for varied sediment transport formulas, Exp. No. 7

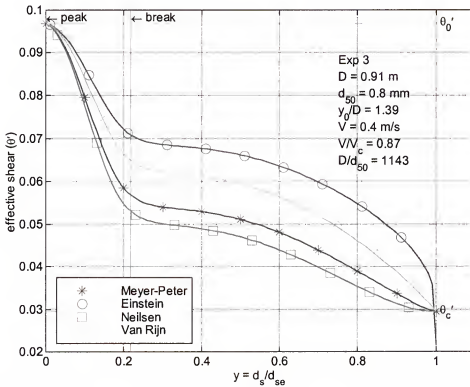


Figure 4-9. Effective shear stress for varied sediment transport formulas, Exp. No. 3

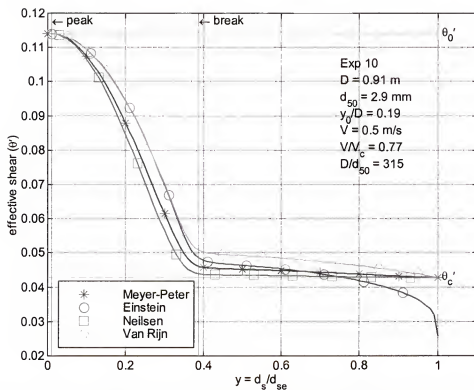


Figure 4-10. Effective shear stress for varied sediment transport formulas, Exp. No. 10

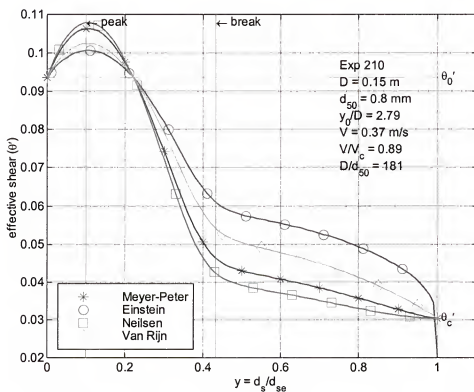


Figure 4-11. Effective shear stress for varied sediment transport formulas, Exp. No. 210

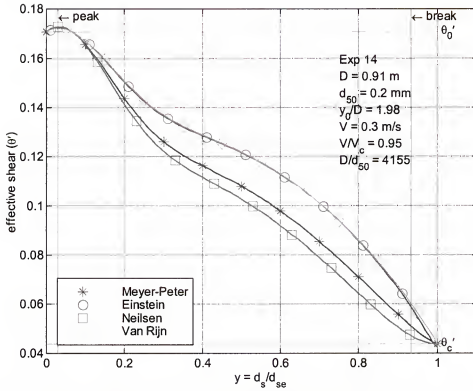


Figure 4-12. Effective shear stress for varied sediment transport formulas, Exp. No. 14

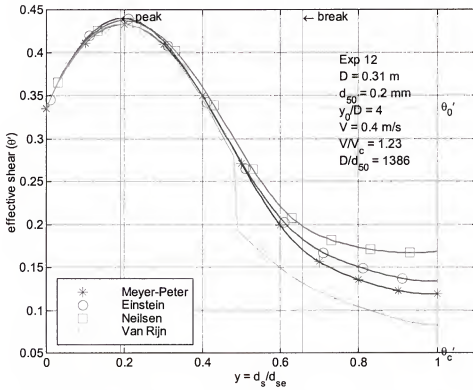


Figure 4-13. Effective shear stress for varied sediment transport formulas, Exp. No. 12

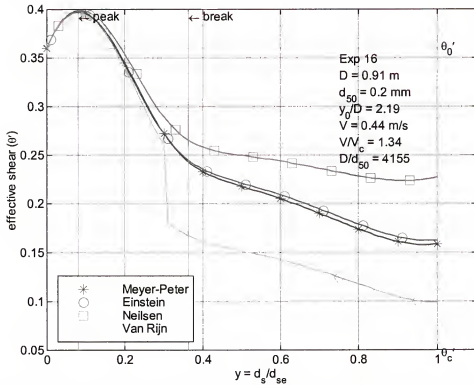


Figure 4-14. Effective shear stress for varied sediment transport formulas, Exp. No. 16

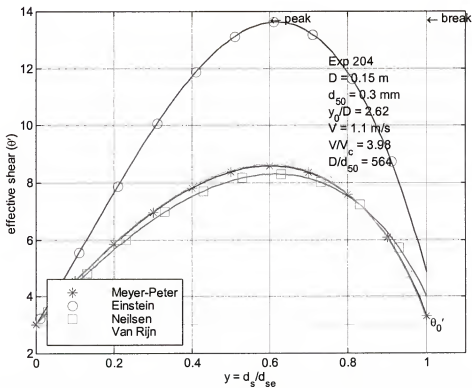


Figure 4-15. Effective shear stress for varied sediment transport formulas, Exp. No. 204

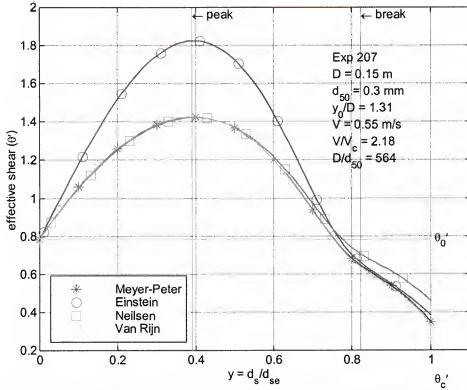


Figure 4-16. Effective shear stress for varied sediment transport formulas, Exp. No. 207

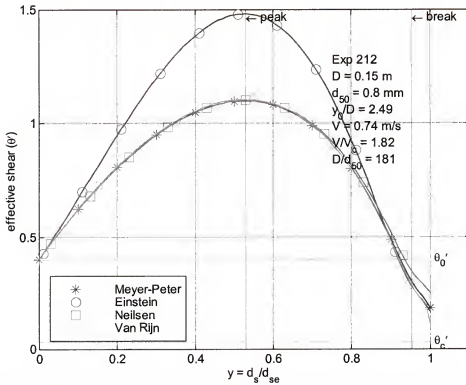


Figure 4-17. Effective shear stress for varied sediment transport formulas, Exp. No. 212

The different forms of the transport equations lead to obvious differences in the profiles of effective shear stress as a function of scour depth. The Van Rijn profiles exhibit discontinuities due to regime change at $T = 3$. The Einstein clear water scour profiles do not end at the critical shear stress since the Einstein bed-load transport does not distinguish between clear water scour and live bed scour conditions (i.e., there is no minimum shear stress for which transport occurs). With the Einstein formula, all experiments have an upstream sediment transport. However, the profiles for each experiment have a common shape with several key common elements.

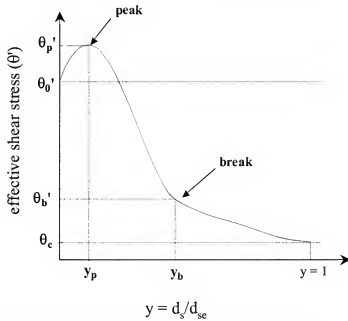


Figure 4-18. Idealized effective shear stress vs. scour depth profile

Figure 4-18 is an idealized profile containing all of the main features. Generally, the effective shear stress shows an initial increase to some peak value. This is followed by a rapid decrease in the effective shear stress as the scour hole deepens. At some scour depth (usually less than 50% of the equilibrium values under the clear water scour condition), there is a sudden change in the slope of the profile. This change will be referred to as the "break point." Following this break, the effective shear stress becomes

fairly linear until the equilibrium scour depth is reached. There are variations in the profiles depending on the specific flow, sediment and structure parameters.

Figures 4-9, 4-10, and 4-12 show examples where only a very small peak or no peak exists. In Figure 4-13 the break is more of a continuous parabolic curve rather than a sharp change in slope. In Figure 4-12 there is essentially no change in slope and in Figure 4-14 there is a second change in slope following the linear period. Figures 4-15, 4-16, and 4-17 are profiles for live bed scour experiments. In these experiments, the peak dominates the profile and the break occurs very close to the equilibrium scour depth or does not occur at all.

Melville (1975), Nakagawa and Suzuki (1975), and Ettema (1980) described the initial phase where the scour begins due to streamline deformation about the cylinder. During this phase, the downflow develops and the horseshoe vortex forms when the downflow curls at the base of the cylinder. The formation of the vortex creates turbulence which enhances the sediment transport by mobilizing the sediment and raising it into the water column. There the sediment can be swept downstream.

Since the model uses a simple bed-load function for the sediment transport, the increased sediment transport due to the establishment of the downflow and the formation of the horseshoe vortex manifests itself as an increase in the effective shear stress in the erosion zone. The fit equations (Eq. 3-1 and 3-2) with their dual time constants when combined with the model (Eq. 4-27) with the incorporated scour hole geometry capture this process quite well for the experiments.

Ettema (1980) described the principal erosion phase where the horseshoe vortex, following a period of growth in size and strength, settles into the scour hole and weakens.

As the scour hole grows, the volume of material avalanching into the hole increases rapidly by the square of the increase in scour depth (Eq. 4-16). The difference between the rapid increase in volume to be transported out of the hole and the decreasing growth rate of the vortex strength appears in the model as a reduction in the effective shear stress with increased scour depth. As the vortex weakens, the effective shear stress decreases with depth and the rate at which material is transported out of the scour hole decreases. At this point the effective shear stress varies nearly linearly with scour depth.

For some flow conditions, a point is reached where the horseshoe vortex is fully contained in the scour hole (Figure 4-10). While the vortex is above the top of the scour hole, energy is transferred to the vortex from the mean flow directly and indirectly through the downflow. When the vortex is finally contained in the scour hole, less energy can be transferred to the vortex directly from the mean flow leaving only the energy from the downflow to add significant energy to the vortex. The "break" in the effective shear stress profile is this reduction in the energy being transferred to the horseshoe vortex.

The break is not necessarily as sharp as in Figure 4-10. For flows with V/V_c greater than approximately 0.85, the change is subtle. In some cases, such as live bed scour, there is no change in the effective shear stress slope. Under live bed scour conditions, the rate of sediment transport into the scour hole increases rapidly with increasing velocity. Consequently, equilibrium conditions may be achieved before the horseshoe vortex is submerged and the sharp reduction in energy transferred to the horseshoe vortex never occurs.

Modeling the Effective Shear Stress Profile

A necessary input to the scour model (Eq. 4-27) is the shear stress as a function of scour depth. This requires a model for the shear stress profile. The key points of the shear stress as a function of scour depth are the initial shear (θ_0') the peak shear stress (θ_p'), the shear stress at the break point (θ_b'), and the shear stress at the equilibrium scour depth (θ_f'). The value of the initial shear stress was discussed previously. The value of the final shear stress is determined from the input sediment transport and the scour transport function. The rest of the model inputs can be determined from the endpoints and the peak and break points.

For both clear water scour and live bed scour conditions, the effective shear stress peak can be described as a concave-down parabola with its vertex at the peak. This parabola is not necessarily symmetrical and two parabolas, joined at a common vertex, are used. The first is constructed from the peak and the initial shear stress. The end point for the second parabola can vary. Live bed experiments have highly symmetrical peaks except when the shear stress peak occurs at a depth greater than 50% of the equilibrium scour depth. For these cases, the peak and the final shear stress are used to construct the second parabola and the model is complete.

If the final shear stress is below the initial shear stress and the peak occurs at a depth greater than $y = 0.25$, the profile generally crosses below the value of the initial shear stress at $y = 2y_p$. The profile below this point is linear from $(2y_p, \theta_0')$ to $(1, \theta_f')$. In this case, the vortex is large compared to the equilibrium scour depth and does not descend far enough into the scour hole to weaken significantly before equilibrium is reached. If the peak occurs at a depth less than $y = 0.25$, the vortex is small and will

begin to weaken before equilibrium is reached. Under live bed scour conditions, the profile is best modeled as a concave-up parabola with its vertex at the equilibrium point $(1, \theta'_f)$. The intersection of the second concave-down parabola and the concave-up parabola can be taken to be $(2y_p, \theta'_0)$.

Under clear water scour conditions, the shear stress crosses below the value of the initial shear stress at some depth $y > 2y_p$ and a break point will exist where $\theta'_b < \theta'_0$. This break is relatively sharp in most cases. The second parabola is best constructed with its endpoint at the break point (y_b, θ'_b) . The shear stress profile can then be approximated by a straight line from the break point to the final shear stress which is taken to be the critical shear stress. Table 4-2 summarizes this model.

Table 4-2. Effective shear stress model summary

Value of y ($= d_s/d_{se}$)	Equation for shear stress (θ')
$0 \leq y \leq y_p$	$\theta' = \theta'_p - (\theta'_p - \theta'_0)[(y - y_p)/y_p]^2$
$y_p < y \leq y_b$	$\theta' = \theta'_p - (\theta'_p - \theta'_b)[(y - y_p)/(y_b - y_p)]^2$
$y_b < y \leq 1$	if $y_p \leq 0.25$ and $\theta_b \geq \theta'_0$ $\theta' = \theta'_f - (\theta'_f - \theta'_b)[(y - 1)/(1 - y_b)]^2$ if $y_p > 0.25$ or $\theta_b < \theta'_0$ $\theta' = \theta'_f + (\theta'_f - \theta'_b)(y - 1)/(1 - y_b)$

Using the Meyer-Peter Mueller transport function, the effective shear stress was calculated using the shear stress model (Table 4-2). The result was compared to the effective shear stress profile calculated by solving Eq. 4-30 for θ' . Figures 4-19 through 4-23 show the results of this comparison. The left plot shows effective shear stress calculated by solving Eq. 4-30 for θ' (solid blue line). The peak and break points from this curve were applied to the shear stress model in Table 4-2 and a model profile was plotted (dashed red line). The right plot shows the data time series (dotted points) and the

curve obtained by solving Eq. 4-27 with the modeled shear stress profile in the left plot.

The plots show that the shear stress model of Table 4-2 can be used in the scour model to produce an accurate scour time series.

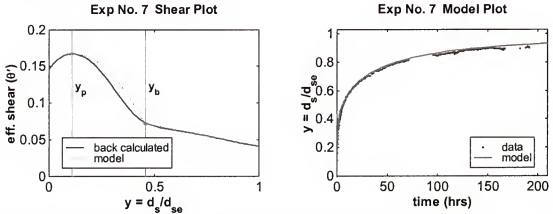


Figure 4-19. Experiment 7, shear stress profile and model result

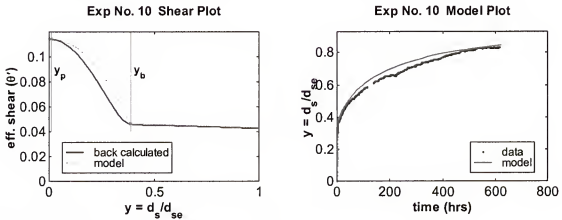


Figure 4-20. Experiment 10, shear stress profile and model result

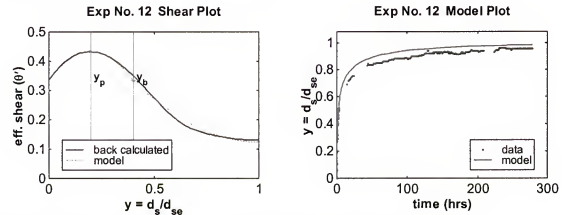


Figure 4-21. Experiment 12, shear stress profile and model result

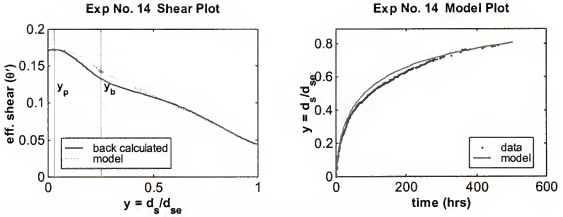


Figure 4-22. Experiment 14, shear stress profile and model result

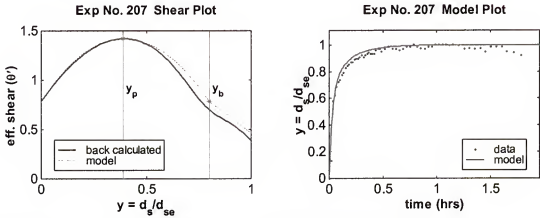


Figure 4-23. Experiment 207, shear stress profile and model result

Determination of the Points for the Shear Stress Model

The final requirement for the practical application of the scour model is to determine the six coefficients or points required as input to the shear stress model. The initial effective shear stress (θ_0') was discussed previously. The remaining coefficients and points are the sediment transport coefficient (C) in Eq. 4-27, the scour depth and the value of the peak effective shear stress (y_p, θ_p'), and the scour depth and value of the effective shear stress at the break point (y_b, θ_b'). The final shear stress (θ_f') is also required. This value may be assumed to be the critical shear stress (θ_c) for clear water scour conditions or may be calculated by solving Eq. 4-27 for θ' at $y = 1$ for live bed

scour conditions.

Also required to complete the model inputs is the equilibrium scour depth (d_{se}). For the experiments, this value is known a priori. However, for application or theoretical studies, this value must be calculated. Over the years, many researchers have developed empirical equations for this value (Laursen and Toch 1956, Neill 1964, Shen et al. 1966, Hancu 1971, Breusers et al. 1977, Garde et al. 1993, Melville 1997, and Richardson and Davis 2001). Sheppard has published a series of local scour equations that have evolved as more and better data was obtained (Sheppard et al. 1995, Sheppard 2002b). These equations have been accepted by the Florida Department of transportation for use in computing local scour at bridge piers in Florida. They are being used by other states as well. The equations of Sheppard (2002b) are used for the example problems presented in Chapter 5 and are given in Appendix C.

The Sediment Transport Coefficient (C)

As discussed in section 4.3, it is assumed that the sediment transport coefficient does not depend on the depth of the scour hole. Using the fitted curves, the initial nondimensional sediment transport rate ($q_0^* = q_{0-out} / \sqrt{(s-1)gd_{50}^3}$) was determined from Eq. 4-27 for each experiment. The values were plotted against a number of parameters to look for dependencies. These parameters included V/V_c , y_0/D , D/d_{50} , VD/v , $V^2/[(sg-1)gd_{50}]$, $V^2/[(sg-1)gD]$, θ_0/θ_c and $(\theta_0' - \theta_c)$. As might be expected, the initial transport rate was strongly dependent on the excess shear stress ($\theta_0' - \theta_c$). Figure 4-24 is a plot of q_0^* as a function of the excess shear stress for the live bed scour range data. The points are numbered by experiment. Referring to Table 3-3 for flow and sediment conditions for each experiment, an additional dependency on D/d_{50} is apparent. The curves in Figure 4-

24 have the form

$$q_0^* = \frac{4(\theta_0' - \theta_c)^{1.5}}{f(D/d_{50})(\theta_0' - \theta_c)^{1.5} + 1} \quad (4-30)$$

Using Eq. 4-31, a value for $f(D/d_{50})$ was calculated for the live bed scour experiments and plotted in Figure 4-25. The function for the curve is

$$f(D/d_{50}) = 0.25[1 + 30\exp(-0.01D/d_{50})].$$

Combining this equation and Eq. 4-31 gives the final equation for the initial sediment transport rate under live bed scour conditions,

$$q_0^* = \frac{4(\theta_0' - \theta_c)^{1.5}}{0.25[1 + 30\exp(-0.01D/d_{50})](\theta_0' - \theta_c)^{1.5} + 1} \quad (4-31)$$

For the live bed scour experiments, the excess shear stress has a range of $0.2 \leq (\theta_0' - \theta_c) \leq 4$. The initial transport rate dependency on the excess shear stress is straightforward. Figure 4-24 indicates that as $(\theta_0' - \theta_c)$ increases, the ability of the flow to mobilize and transport sediment increases causing the transport rate to increase.

The range of D/d_{50} is $181 \leq D/d_{50} \leq 4155$ for the live bed scour experiments. Figure 4-25 shows the dependency of the initial transport rate on D/d_{50} . For large D/d_{50} , the initial transport appears independent of D/d_{50} . For small D/d_{50} , the $f(D/d_{50})$ from Eq. 4-31 drops sharply with increasing D/d_{50} , indicating an increase in the initial transport for increasing D/d_{50} . Increasing the cylinder diameter increases the Reynolds Number of the cylinder. An increased Reynolds number generally indicates increased turbulence. More turbulence in the flow will mobilize more sediment and increase the ability or efficiency of the excess shear stress to transport sediment. Therefore as D/d_{50} increases due to an increase in the cylinder diameter, the initial transport rate will increase. As D/d_{50}

increases due to decreasing sediment size, the sediment is also more easily mobilized and the transport increases as well.

At some point, this effect of increasing D/d_{50} appears to reach a limit. This is likely due to a limit in the amount of sediment that can be mobilized into the water column. However, it should be noted that this limit is based on experiments 12, 13 and 16 in Figure 4-24. The excess shear stress values for these experiments are at the point of convergence of the curves and so the limit with respect to D/d_{50} indicated in Figure 4-25 may be incorrect. More data for higher values of $(\theta_0' - \theta_c)$ with prototype size values of D/d_{50} is needed before this issue can be resolved.

Figure 4-26 plots the initial transport rate calculated with Eq. 4-32 versus that calculated using the time series for each live bed scour experiment and Eq. 4-27.

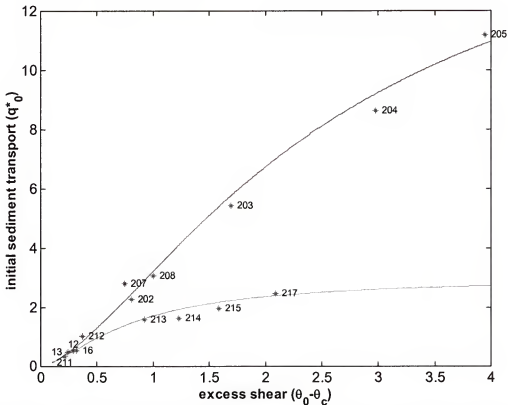


Figure 4-24. Initial sediment transport as a function of excess shear stress for live bed scour experiments

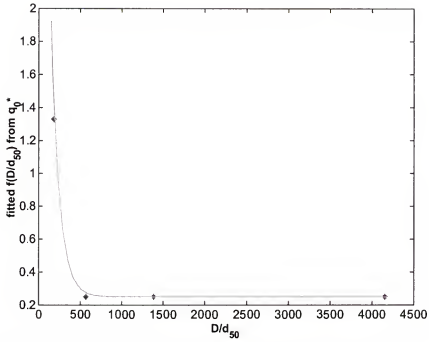


Figure 4-25. Fitted D/d_{50} function for Eq. 4-31, live bed scour experiments

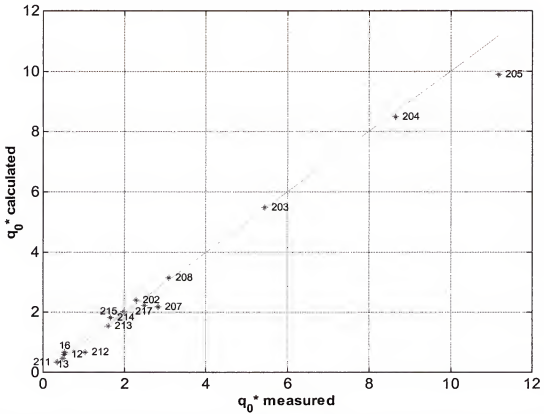


Figure 4-26. Calculated initial transport vs. initial transport derived from experiment fit curves (live bed scour)

Figure 4-27 is a plot of the initial transport rate as a function of the excess shear stress for the clear water scour experiments. The clear water scour data has more scatter than the live bed scour experiments, indicating that the lower flow intensity allows other parameters to influence the transport. As previously noted, some of these experiments were subject to suspended fine sediment effects. This would lower the transport rate. The curve in Figure 4-27 has the form of Eq. 4-31 with the coefficient valued at 4 taking a value of 8. Figure 4-28 plots $f(D/d_{50})$ versus D/d_{50} for this data. Solving for $f(D/d_{50})$ as was done for the live bed scour experiments gives

$$q_0^* = \frac{8(\theta_0' - \theta_c)^{1.5}}{10[1 + 30 \exp(-0.01D/d_{50})](\theta_0' - \theta_c)^{1.5} + 1} \quad (4-32)$$

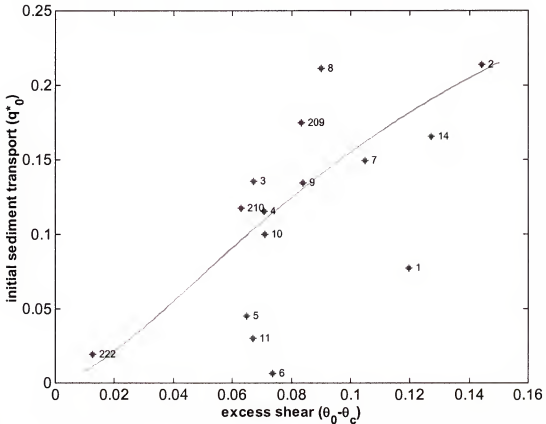


Figure 4-27. Initial sediment transport as a function excess shear stress for clear water scour experiments

Eq. 4-33 shows the same relationships as the live bed scour equation; however the effect of increasing the turbulence by increasing D/d_{50} is greater. As with the live bed scour data, the scarcity of data points for large D/d_{50} makes it difficult to determine the effect of increasing D/d_{50} beyond approximately 500.

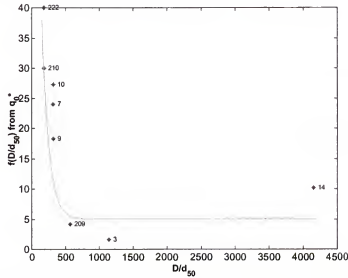


Figure 4-28. Fitted D/d_{50} function for Eq. 4-31, clear water scour experiments

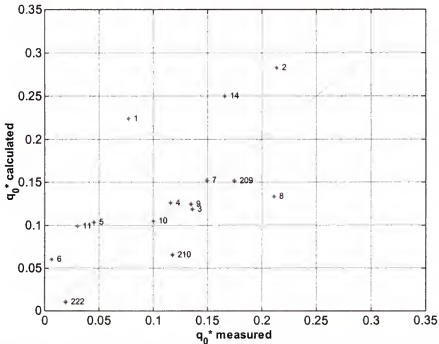


Figure 4-29. Calculated initial transport vs. measured initial transport derived from experiment fit curves (clear water scour).

Figure 4-29 plots the initial transport rate determined from Eq. 4-33 versus that calculated using the fitted time series for each clear water scour experiment. Again, there is scatter in this data which may be due to the reduction in initial transport rate caused by the effects of suspended fine sediments.

Using the initial transport rate calculated with Eq. 4-32 and 4-33, the transport coefficient in Eq. 4-27 can be calculated from

$$C = \frac{q_0^* \sqrt{(sg-1)gd_{50}^3}}{f(\theta_0')}$$

The form of the transport function in Eq. 4-32 and 4-33 is that of Meyer-Peter Mueller. Therefore, using a Meyer-Peter Mueller transport function, the transport coefficient becomes

$$C = \frac{A\sqrt{(sg-1)gd_{50}^3}}{B[1 + 30\exp(-0.01D/d_{50})](\theta_0' - \theta_c')^{1.5} + 1} \quad (4-33)$$

where $A = 4$ and $B = 0.25$ for $V/V_c \geq 1$ and $A = 8$ and $B = 10$ for $V/V_c < 1$.

Peak Effective Shear Stress (y_p, θ_p')

Using the transport coefficient from Eq. 4-34, Eq. 4-27 was solved for θ' to construct an effective shear stress profile for each experiment. The peak and break points were adjusted to give the model shear stress input for Eq. 4-27 that gave the most accurate reproduction of the experimental time series. These points ($y_p, \theta_p', y_b, \theta_b'$) were plotted individually against the same parameters as the initial sediment transport rate.

For the live bed scour experiments, the scour depth at which the peak shear stress occurs showed a strong dependency on the flow intensity (V/V_c). Figure 4-30 is a plot of the scour depth at which the shear stress peak occurs for the live bed scour experiments

as a function of the flow intensity. The plot shows a distinct division in the live bed region at $V/V_c = 2$. Above this point, the depth of the shear stress peak does not exhibit a dependency on D/d_{50} for the range $181 \leq D/d_{50} \leq 564$. Beyond this D/d_{50} range, data is not available. A fit curve for y_p as a function of V/V_c only is plotted in Figure 4-30.

Below $V/V_c = 2$, the dependence on the flow intensity changes and a dependence on D/d_{50} appears. Figure 4-31 shows this plot at the lower flow intensities and indicates an effect from D/d_{50} . The result is in a family of curves of the form

$$y_p = f(D/d_{50})(V/V_c - \epsilon)^{1.5}.$$

Recall that ϵ is the minimum value of V/V_c required for scour to initiate (approximately $V/V_c = 0.47$). Figure 4-32 is a plot of $f(D/d_{50})$ versus D/d_{50} for this equation.

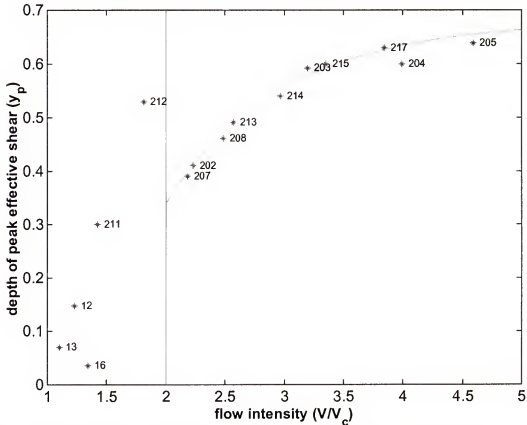


Figure 4-30. Depth of peak shear stress as a function of V/V_c for live bed scour

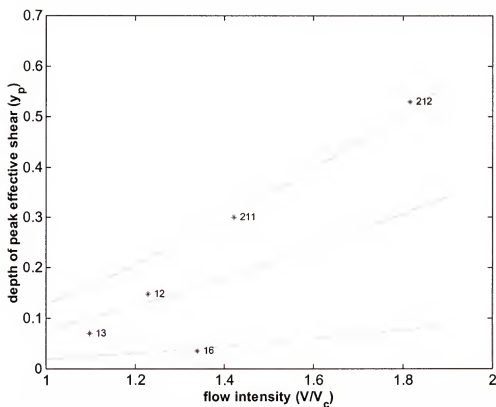


Figure 4-31. Depth of the peak shear stress as a function of V/V_c for $1 \leq V/V_c \leq 2$

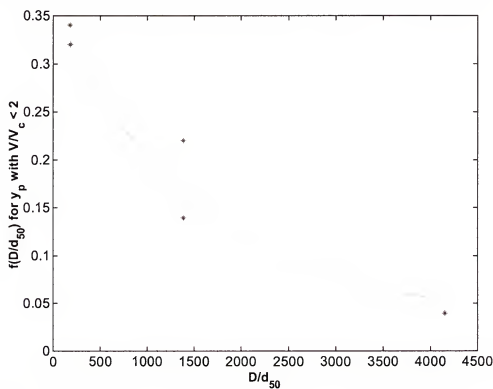


Figure 4-32. Variation of the V/V_c function for y_p with D/d_{50} for $1 \leq V/V_c \leq 2$

Accounting for the D/d_{50} effect in the lower flow intensity region, the fit equations for the depth of the peak shear stress under live bed scour conditions are

$$y_p = 0.68 \left[1 - 3.7 \exp\left(-\frac{V}{V_c}\right) \right] \text{ for } V/V_c > 2 \quad (4-34)$$

$$\text{and } y_p = 0.035 \left(\frac{V}{V_c} - \varepsilon \right)^{1.5} \left[1 + 10 \exp\left(-\frac{D/d_{50}}{1500}\right) \right] \text{ for } 1 \leq V/V_c \leq 2. \quad (4-35)$$

Figure 4-33 is a plot of the scour depth at which the effective shear stress peak occurs calculated from Eq. 4-35 and 4-36 versus the "measured" value. The "measured" value is that obtained by solving Eq. 4-27 for θ' for the live bed scour experiments.

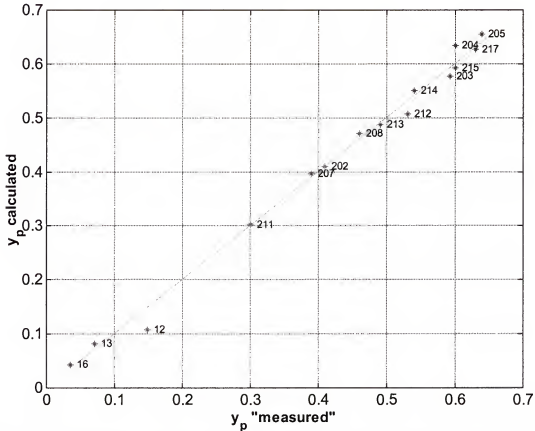


Figure 4-33. Calculated vs. "measured" depth of the peak shear stress for live bed scour experiments

Again, the data for the clear water scour tests shows more scatter. However,

Figure 4-34 indicates that the scour depth at which the peak shear stress occurs shows the

same dependencies on V/V_c and D/d_{50} in clear water scour as in live bed scour. Using experiments 7, 8, 9, 10 and 11, an additional dependency on y_0/D was determined. When y_0/D is less than 1, increasing y_0/D increases the depth of the shear stress peak. Above $y_0/D = 1$, y_p is independent of y_0/D . Along with the data, Figure 4-34 is a plot of a family of curves of the form

$$y_p = f\left(\frac{D}{d_{50}}\right) f\left(\frac{y_0}{D}\right) \left[\frac{V}{V_c} - \varepsilon \right]^{0.8}.$$

Figures 4-35 and 4-36 illustrate the D/d_{50} and y_0/D dependencies.

Adding the functions from Figures 4-35 and 4-36 gives the final equation for y_p under clear water scour conditions,

$$y_p = 0.023 \left(\frac{V}{V_c} - \varepsilon \right)^{0.8} \left[1 + 10 \exp\left(-\frac{D/d_{50}}{1500} \right) \right] \tanh \left[4 \left(\frac{y_0}{D} \right)^2 \right]. \quad (4-36)$$

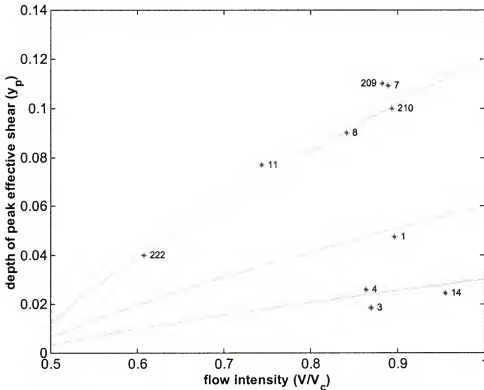


Figure 4-34. Depth of the peak shear stress as a function of V/V_c for $V/V_c < 1$

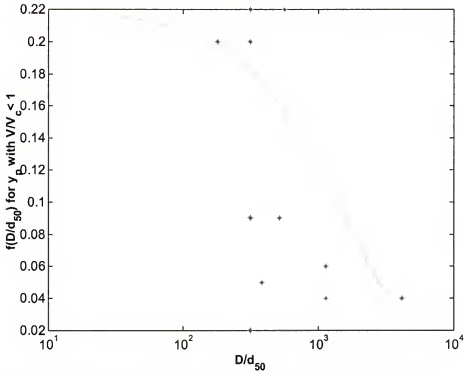


Figure 4-35. Variation of the V/V_c function for y_p with D/d_{50} for $V/V_c \leq 1$

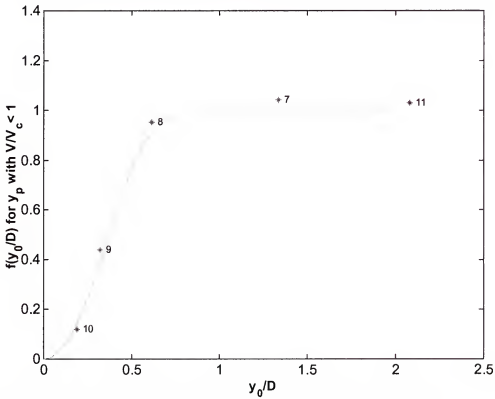


Figure 4-36. Variation of the V/V_c function for y_p with y_0/D for $V/V_c \leq 1$

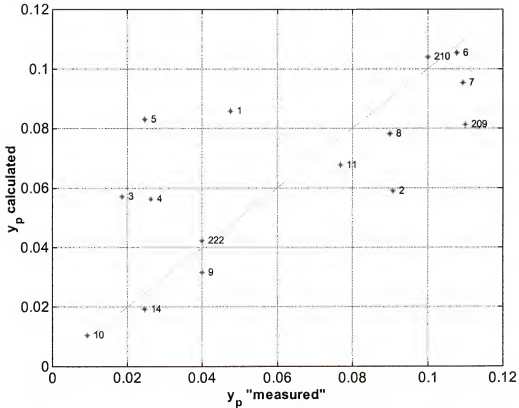


Figure 4-37. Calculated vs. "measured" depth of the peak shear stress for clear water scour experiments

Figure 4-37 is a plot of the depth of the shear stress peak calculated with Eq. 4-37 versus the "measured" value for the clear water scour experiments.

Previously, it was hypothesized that the effective shear stress peak occurs at the point where there is a balance between the increasing volume of material to be transported out of the hole and the decreasing growth rate of the horseshoe vortex. Thus, anything that increases the size and strength of the horseshoe vortex will tend to allow the flow to mobilize and transport a larger volume of material and shift the balance point to a deeper scour depth.

The "horseshoe vortex is formed when the boundary layer on the bed upstream of the (cylinder) undergoes a three dimensional separation under the influence of the adverse pressure gradient induced by the structure" (Sumer and Fredsoe, 2002, p. 151).

Figure 4-38 shows the separation line (S) and the influence of the distance of the separation point (x_s) from the cylinder on the size of the vortex. The separated boundary layer forms the horseshoe vortex and trails off downstream.

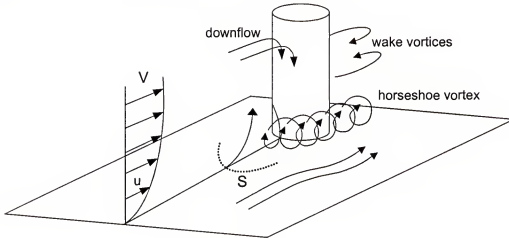


Figure 4-38. The separation line associated with formation of the horseshoe vortex

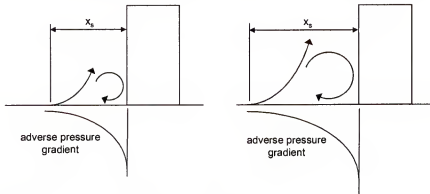


Figure 4-39. Effect of the adverse pressure gradient

To examine Eq. 4-35, 4-36 and 4-37, the variation of each parameter in the equations is considered while maintaining the other parameters constant. First, consider the flow intensity. From potential theory, the strength of the horseshoe vortex is proportional to its circulation (Γ). Figures 4-38 and 4-39 show the effect of the adverse pressure gradient due to the presence of the cylinder on the circulation. The circulation is a function of the upstream velocity (V) and the size of the vortex. The size of the vortex

is dependent on the location of the separation point (x_s), and, therefore the circulation is a function of the product of the upstream velocity and the distance of the separation point from the cylinder, $\Gamma = f(Vx_s)$. Increasing the flow intensity by increasing the flow velocity will increase the strength of the vortex and so increase the depth of the peak shear stress by slowing the decrease in the growth rate of the vortex system.

From potential theory, it can be shown that the pressure gradient upstream of the cylinder varies as V^2/D , decreasing as the distance upstream from the cylinder increases (Figure 4-39). This is the adverse pressure gradient which causes the boundary layer to separate. As the upstream flow intensity increases, the adverse pressure gradient increases and separation occurs further upstream (i.e., x_s will increase). This causes the vortex to become larger, increases its strength and increases the depth of the peak shear stress.

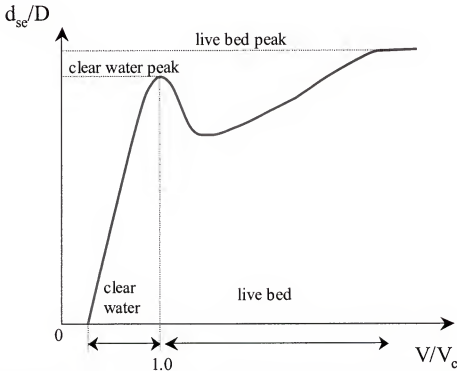


Figure 4-40. Variation of normalized equilibrium scour depth (d_{sc}/D) with flow intensity (V/V_c) (adapted from Gosselin, 1997)

The effect of the flow intensity on the equilibrium scour depth opposes these effects. Figure 4-40 shows that as V/V_c increases, the equilibrium scour depth generally increases. Increasing the equilibrium scour depth tends to decrease the normalized depth of the shear stress peak (y_p) which is the scour depth at which the shear stress peak is reached divided by the equilibrium scour depth. Figures 4-30, 4-31 and 4-34 show that y_p increases with V/V_c but at a decreasing rate. The decreasing rate may be attributed to the increase in equilibrium scour depth with increasing V/V_c . This is the likely source of the y_p limit indicated in Figure 4-30 and Eq. 4-35.

The exception occurs for V/V_c just greater than 1. At these flow intensities, increasing V/V_c tends to decrease the equilibrium scour depth. This is reflected in the discontinuity noted in Figure 4-30 for $V/V_c < 2$. For $1 < V/V_c < 2$, all increasing flow intensity effects tend to increase y_p and the result is the very high value of y_p for experiment 212 in Figure 4-30.

From Eq. 4-36, increasing D/d_{50} reduces the depth at which the peak shear stress occurs for $V/V_c < 2$. Increasing the cylinder diameter (D) will increase the equilibrium scour depth and decrease the normalized depth of the shear stress peak (y_p). From potential theory, increasing D will decrease the adverse pressure gradient, causing separation to occur closer to the cylinder, reducing the size and strength of the horseshoe vortex and decreasing the depth at which the shear stress reaches its peak value.

The turbulence induced by the presence of the cylinder has a similar effect. As D increases, VD/v increases and the general turbulence in the vicinity of the cylinder increases. The increased turbulence causes a greater exchange of momentum between the fluid layers which delays the separation (i.e., closer to the cylinder). The delayed

separation results in a reduction in the size and strength of the horseshoe vortex and decreases the depth at which the shear stress reaches its peak value.

A decrease in D/d_{50} due to a decrease in sediment size means that the critical velocity (V_c) will be reduced. This requires a corresponding decrease in the upstream flow velocity to maintain flow intensity constant. This will result in a weaker horseshoe vortex and a shallower scour depth for the peak shear stress. In general, the scour depth at which the effective shear stress peak occurs is reached sooner relative to the equilibrium scour depth (i.e., at a lower normalized scour depth, y_p) as D/d_{50} is increased.

Melville and Chiew (1999) explained that a surface roller or bow wave forms at the leading edge of the cylinder. This roller rotates opposite to the horseshoe vortex. For shallow aspect ratios (y_0/D), it can reduce strength of the horseshoe vortex. Therefore, a lower y_0/D will reduce the depth of the peak shear stress as shown in Eq. 4-37. For aspect ratios greater than 1, the surface roller no longer influences the horseshoe vortex and the depth of the shear stress peak is independent of y_0/D .

Experiments 12 and 13 have aspect ratios of 4 and 0.6, respectively. Figures 4-31 and 4-33 show a lack of dependence of y_p on y_0/D for these experiments. This indicates that the flow intensity dominates the surface roller effect under live bed scour conditions.

The magnitude of the peak effective shear stress (θ_p') for the live bed scour experiments are plotted as a function of the initial shear stress (θ_0') in Figure 4-41. The plot shows that θ_p' is strongly dependent on the θ_0' . However, the data with flow intensities below 2 ($1 \leq V/V_c \leq 2$) is not in good agreement with the fitted curve (experiments 24, 25, 26, 211, 212). At these lower flow intensities, y_p is small (less than 0.25). It is reasonable to conclude that θ_p' will approach θ_0' as the depth at which the

peak is reached approaches zero. Therefore, y_p was incorporated into the fit equation for θ_p' . The result provided good agreement with the data (Figure 4-42).

The equations for the fit curves in Figures 4-41 and 4-42 are

$$\theta_p' = 2.2(\theta_0')^{1.2} \text{ for } V/V_c > 2 \quad (4-37)$$

$$\text{and } \theta_p' = \theta_0' + 23(y_p \theta_0')^2 \text{ for } 1 \leq V/V_c \leq 2. \quad (4-38)$$

Figure 4-43 is a plot of the peak shear stress calculated with Eq. 4-38 and 4-39 versus the "measured" peak shear stress. Again, the "measured" shear stress was found by solving Eq. 4-27 for θ' given the fitted rate of change of normalized scour depth (dy/dt) for each of the live bed scour experiments.

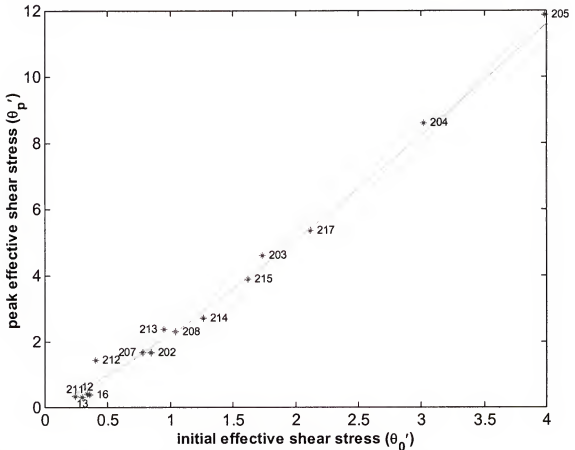


Figure 4-41. Peak shear stress as a function of initial shear stress for $V/V_c > 1$

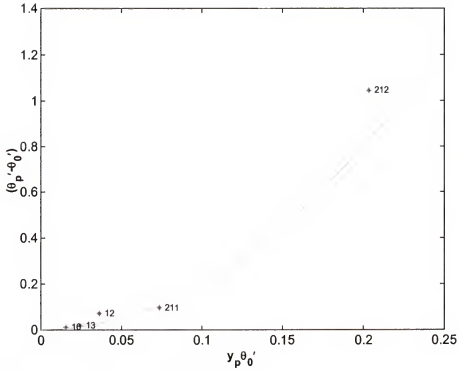


Figure 4-42. Peak shear stress for $1 \leq V/V_c \leq 2$

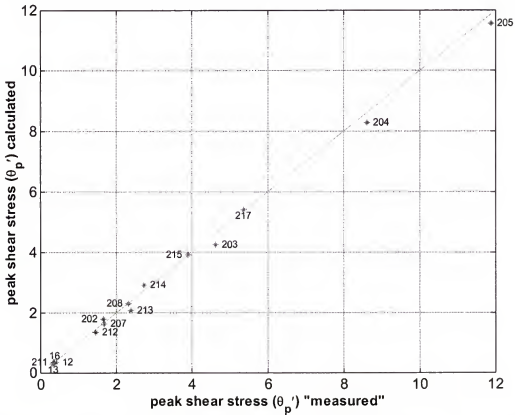


Figure 4-43. Calculated peak shear stress vs. "measured" for live bed scour experiments

Extending the relationship of Eq. 4-39 into the clear water scour range, $(\theta_p' - \theta_0')$ versus $y_p \theta_0'$ is plotted in Figure 4-44 for the clear water scour experiments.

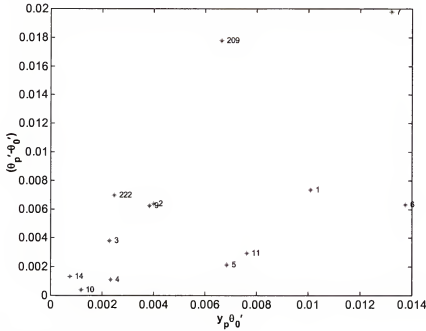


Figure 4-44. Peak shear stress for $V/V_c < 1$

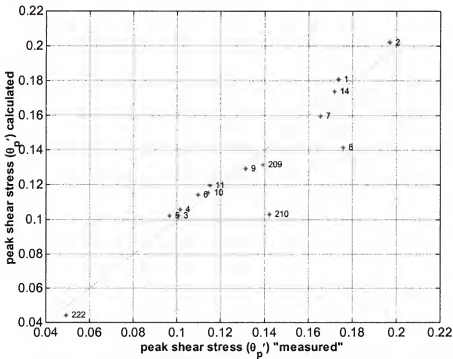


Figure 4-45. Calculated peak shear stress vs. "measured" for clear water scour experiments

Though the data was scattered, an attempt was made to fit $(\theta_p' - \theta_0')$ to $y_p \theta_0'$ in a linear relationship. The best fit relationship for these experiments is

$$\theta_p' = \theta_0' (1 + y_p) \quad (4-39)$$

Figure 4-45 is a plot of the peak shear stress calculated with Eq. 4-40 versus "measured" values. The results are surprisingly good given the scatter of the data in Figure 4-44.

Eq. 4-38, 4-39 and 4-40 indicate that the peak shear stress increases with increasing initial shear stress. Since the initial shear stress was determined to be proportional to the upstream shear stress, as the flow velocity increases so does the peak shear stress. This is reasonable since the strength of the horseshoe vortex is directly dependent on the upstream flow intensity and the effective shear stress is dependent on the ability of the horseshoe vortex to mobilize and transport sediment.

For lower flow intensities ($V/V_c < 2$), the effect of the upstream flow is less dominant and the scour depth at which the peak occurs influences the magnitude of the peak. Therefore, Eq. 4-39 and 4-40 indicate that the effects of D/d_{50} and y_0/D (that lead to a stronger horseshoe vortex and an increased y_p) will result in a greater effective peak shear stress. Generally, decreasing D/d_{50} and increasing y_0/D will increase the peak effective shear stress.

At approximately $y_0/D = 1$, the peak shear stress becomes independent of y_0/D . As with the depth of the peak shear stress, this dependency can be attributed to the interaction between the surface roller and the horseshoe vortex. At shallow depths the surface roller opposes the horseshoe vortex and tends to reduce its strength and the value of the peak shear stress. At flow aspect ratios greater than one, the surface roller is too far from the horseshoe vortex to influence the peak shear stress.

Break Point (y_b, θ_b')

The live bed scour experiments generally do not have very distinct breaks in the shear stress profile slope. When there is a break, the break point shear stress value is very close to the initial shear stress. In these experiments the shear stress profile may also be completely dominated by the peak and show no break in slope at all, decreasing directly from the peak to the final shear stress in a parabola. In all live bed scour experiments, the depth of the peak shear stress tends to determine the depth at which the break point occurs. If y_p is less than 0.5, the break point occurs at approximately $y = 2y_p$ and the shear stress at the break point has the value of the initial shear stress ($\theta_b' = \theta_0'$). If y_p is greater than 0.5, the symmetry of the shear stress peak requires that no break occurs and the shear stress follows a parabolic profile from the peak shear stress (θ_p') at $y = y_p$ to the final shear stress (θ_f') at $y = 1$. This relationship shows good agreement with the data and allows the time series to be accurately modeled for the live bed scour experiments (see Appendix B for the plots of the model time series versus the data).

The clear water scour experiments tend to have a very distinct break in the shear stress profile slope. The value of the break point shear stress tends to be much less than the initial shear stress. As discussed previously, the break is thought to be caused by a sharp reduction in the energy transferred to the horseshoe vortex resulting in a sharp reduction in the rate of sediment transport out of the scour hole. The reduction in energy transferred to the vortex occurs when the vortex is fully contained within the scour hole and its primary source of energy becomes the downflow. The downflow itself is induced by the vertical stagnation pressure gradient along the nose of the cylinder resulting from the vertical velocity profile of the main flow.

The stagnation pressure on the nose of the cylinder at any height above the bed (z) can be determined from Bernoulli's Equation,

$$\frac{p_s}{\rho g} \sim \frac{u^2(z)}{2g} + z.$$

Integrating the logarithmic velocity profile, $u(z) = V$ at $z = 0.36y_0$ where V is the depth averaged velocity and y_0 is the water depth. The no slip condition requires that the velocity is zero at some point above the bed ($z = z_0$). Therefore, the pressure difference between these two points is roughly

$$\frac{p_s(0.36y_0) - p_s(z_0)}{\rho g} \sim \frac{V^2 - 0}{2g} + 0.36y_0 - z_0$$

or since $y_0 \gg z_0$,

$$\frac{\Delta p_s}{\rho g} \sim \frac{V^2}{2g} + 0.36y_0.$$

Dividing by $0.36y_0$ gives a rough approximation of the pressure gradient which creates the downflow,

$$\frac{\Delta p_s}{\rho g y_0} \sim \frac{V^2}{g y_0} + 1 \text{ which gives } \frac{\Delta p}{\Delta z} \sim \frac{V^2}{y_0}.$$

Since the downflow developed from this vertical pressure gradient is a major contributor of the energy that forms and sustains the horseshoe vortex, the size and strength of the vortex should vary as the vertical pressure gradient, or as V^2/y_0 . A nondimensional form of this parameter can be obtained by using the square of the Froude number based on depth, $V^2/(gy_0)$. This use of the Froude number is only to nondimensionalize V^2/y_0 and does not imply a dependency on the ratio of the flow velocity to the shallow water wave speed normally associated with the Froude number.

Figure 4-46 is a plot of the scour depth of break point as a function of $V^2/(gy_0)$.

The plot shows a strong relationship between the scour depth of break point and $V^2/(gy_0)$. This confirms the relationship between the size and strength of the horseshoe vortex and the scour depth of break point and adds strength to the hypothesis that the shear stress slope break occurs when the vortex is fully contained in the scour hole. The formula for the curve in Figure 4-46 is

$$y_b = y_p + 0.37 \tanh \left[25 \left(\frac{V^2}{gy_0} \right)^{0.8} \right]. \quad (4-40)$$

Figure 4-47 plots the break point depth calculated from Eq. 4-41 versus the "measured" value obtained from Eq. 4-27 for the clear water scour experiments.

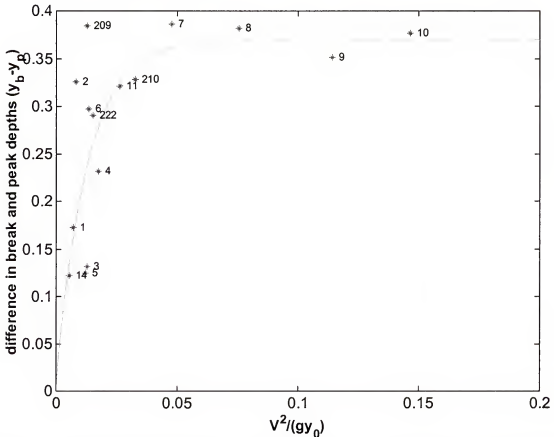


Figure 4-46. Difference between the shear stress break point and peak shear stress depths as a function of $V^2/(gy_0)$

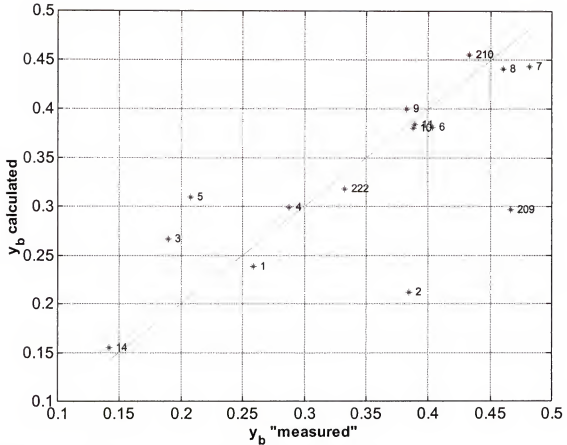


Figure 4-47. Calculated vs. "measured" depth to break point

Since the depth at which the break point occurs has proven to be a function of the vortex size and strength, the shear stress at this break point should also be a function of the strength of the vortex. Figure 4-48 is a plot of the negative of the slope of the final line in the shear stress profile, $(\theta_b' - \theta_c)/(1 - y_b)$, as a function of the sediment number, $N_s^2 = V^2/[(sg-1)gd_{50}]$, for all the clear water experiments.

Except for a few outlying points, the relationship seems to be good. Recall from Chapter 3 and Table 3-4 that experiments 1 and 2 were determined to have been affected by suspended fine sediments. Consequently, it is expected that the value of the sediment transport at the break point for these tests should be artificially low. As a result, the value of the break point shear stress is low.

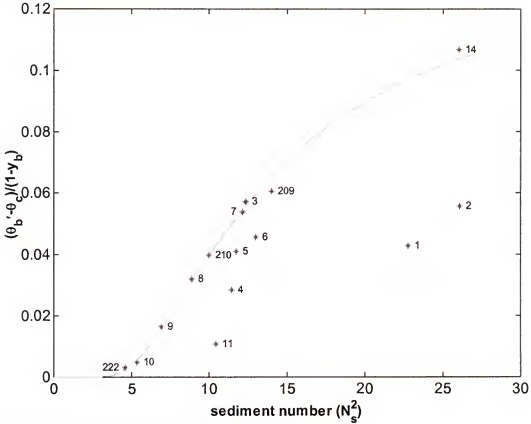


Figure 4-48. $(\theta'_b - \theta_c) / (1 - y_b)$ vs. sediment number (N_s^2)

Recall from the discussions of y_p and θ'_p that increasing D/d_{50} tends reduce the strength and size of the horseshoe vortex and thus reduce the peak effective shear stress (θ'_p). It may be expected that the value of the shear stress at the break point will be similarly affected by varying D/d_{50} . The function plotted in Figure 4-48 is

$$\frac{\theta'_b - \theta_c}{1 - y_b} = 0.14 \exp\left(\frac{-40}{N_s^3}\right).$$

Modifying this equation to incorporate a function of D/d_{50} gives

$$\frac{\theta'_b - \theta_c}{1 - y_b} = f\left(\frac{D}{d_{50}}\right) \exp\left(\frac{-40}{N_s^3}\right).$$

Figure 4-49 is a plot of the D/d_{50} function versus D/d_{50} in the modified equation.

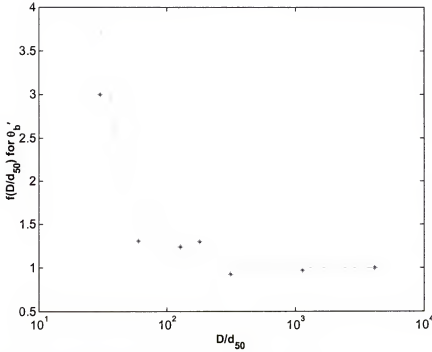


Figure 4-49. Plot of $f(D/d_{50})$ for break point shear stress (θ_b')

Experiments 7, 8, 9 and 10 provided information on the affect of the aspect ratio (y_0/D) on the values of y_p and θ_p' . These experiments all plotted on the curve in Figure 4-48. This indicates that the flow aspect ratio does not affect the break point shear stress. Combining the functions in Figures 4-48 and 4-49 and solving for the break point shear stress gives

$$\theta_b' = \theta_c + 0.14(1 - y_b) \exp\left(\frac{-40}{N_s^3}\right) \left[1 + \left(\frac{D/d_{50}}{50}\right)^{-1.5}\right]. \quad (4-41)$$

Figure 4-50 is a plot of θ_b' calculated with Eq. 4-42 versus the "measured" values of θ_b' for all of the clear water scour data. Agreement is generally good. Though the calculations for experiments 1, 2, 4, 5, 6, 11 and 209 all overestimate the break point shear stress, these experiments have varied D/d_{50} (143 to 1142) and deep aspect ratios (i.e., $y_0/D > 1$). There does not appear to be any common parameter in these experiments. However, since all of these experiments are overestimated, the calculation

is conservative (i.e., the scour hole will tend toward equilibrium faster). Table 4-3 summarizes the equations developed in this section.

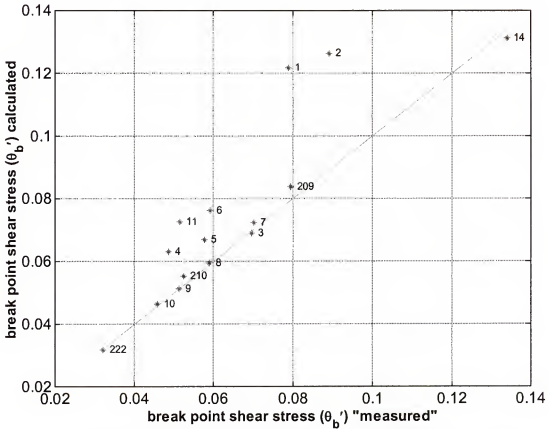


Figure 4-50. Calculated vs. "measured" break point shear stress (θ_b') for clear water scour experiments

4.4 Modeling the Scour Hole Time Series

Using the effective shear stress model described in Tables 4-2 and 4-3, the scour model of Eq. 4-27 can be solved numerically. The method used in this dissertation is an adaptive step size, fifth order Runge-Kutta solution scheme with an embedded fourth order equation for error checking. The numerical solution method and the input data and calculations required to run the model are described in Appendix C.

The input sediment transport rate was estimated for live bed scour conditions by averaging the bed-load functions of the Meyer-Peter Mueller, Nielsen and Van Rijn

formulas in Table 4-1 for the upstream shear stress. The Einstein formula was not used to ensure that there was no input transport for the clear water scour experiments.

Table 4-3. Summary of shear stress model point equations.

Point	V/V_c range	Equation
C	$V/V_c \geq 1$	$C = \frac{4\sqrt{(sg-1)gd_{50}^3}}{0.25[1 + 30\exp(-0.01D/d_{50})](\theta_0' - \theta_c')^{1.5} + 1}$
	$V/V_c < 1$	$C = \frac{8\sqrt{(sg-1)gd_{50}^3}}{10[1 + 30\exp(-0.01D/d_{50})](\theta_0' - \theta_c')^{1.5} + 1}$
y_p	$V/V_c > 2$	$y_p = 0.68 \left[1 - 3.7 \exp\left(-\frac{V}{V_c}\right) \right]$
	$1 \leq V/V_c \leq 2$	$y_p = 0.035 \left(\frac{V}{V_c} - \varepsilon \right)^{1.5} \left[1 + 10 \exp\left(-\frac{D/d_{50}}{1500}\right) \right]$
	$V/V_c < 1$	$y_p = 0.023 \left(\frac{V}{V_c} - \varepsilon \right)^{0.8} \left[1 + 10 \exp\left(-\frac{D/d_{50}}{1500}\right) \right] \tanh \left[4 \left(\frac{y_0}{D} \right)^2 \right]$
θ_p'	$V/V_c > 2$	$\theta_p' = 2.2(\theta_0')^{1.2}$
	$1 \leq V/V_c \leq 2$	$\theta_p' = \theta_0' + 23(y_p \theta_0')^2$
	$V/V_c < 1$	$\theta_p' = (1 + y_p) \theta_0'$
y_b	$V/V_c \geq 1$	If $y_p < 0.5$, $y_b = 2y_p$ If $y_p \geq 0.5$, $y_b = 1$
	$V/V_c < 1$	$y_b = y_p + 0.37 \tanh \left[25 \left(v^2 / gy_0 \right)^{0.8} \right]$
θ_b'	$V/V_c \geq 1$	If $y_p < 0.5$, $\theta_b' = \theta_0'$ If $y_p \geq 0.5$, $\theta_b' = \theta_0'$
	$V/V_c < 1$	$\theta_b' = \theta_c + 0.14(1 - y_b) \exp \left(\frac{-40}{N_s^3} \right) \left[1 + \left(\frac{D/d_{50}}{50} \right)^{-1.5} \right]$

The results of the model solution for each of the clear water and live bed scour experiments listed in Chapter 3 are plotted with the experimental data in Appendix B. For purposes of comparison, predictions using the equations from the Melville and Chiew model (Melville and Chiew 1999) are also plotted. Each plot shows the scour hole depth

normalized with the equilibrium scour depth as a function of time in hours. The model developed here is designated "model" and the Melville and Chiew model is designated "Melville."

Figures B-1 through B-15 plot the clear water scour experiments. Agreement is mixed in this range. Figures B-3, B-7 through B-10 and B-12 through B-14 show good to excellent agreement. Figures B-1, B-2, B-4 through B-6 and B-11 show varying degrees of overprediction while Figure B-15 shows a slight underprediction.

Experiments 1, 2, 5 and 6 (Figures B-1, B-2, B-5, and B-6, respectively) were all noted to have suspended fine sediment effects (Table 3-4). Since these effects reduce the sediment transport in the scour hole, the experimental data would be expected to approach equilibrium slower than the model. This is what the plots show. The effect is especially apparent in experiment 2 (Figure B-2). The model agrees well with the data up to approximately 10 hours where suspended fine sediments cause a drop off in the time series data followed by a sharp cut-off at approximately 55 hours. This suggests that the model may be predicting the time series as it would have occurred had the fine sediments not been present. More data is needed to verify this hypothesis.

The model grossly over predicts experiments 4 and 11 (Figures B-4 and B-11, respectively). In both cases the predicted break point shear stress was too high. The only significant difference between the conditions for experiments 3 (Figure B-3) and 4 (Figures B-4) is the flow depth. The aspect ratio is 1.39 for experiment 3 and 0.95 for experiment 4. Otherwise the experiments were conducted with the same size cylinder and sediment (0.914 m and 0.84 mm, respectively) and nearly the same flow intensities (0.87 and 0.86, respectively). The lower flow aspect ratio of experiment 4 suggests a

surface roller effect which is not accounted for in Eq. 4-42. A surface roller should reduce the strength of the horseshoe vortex and so reduce the break point shear stress for low aspect ratios. The model equations do not include such an effect and so the overprediction is expected.

Experiment 11 continued the series of experiments with a cylinder diameter of 0.914 m and a sediment size of 2.9 mm (experiments 7, 8, 9, 10, and 11). Experiment 11 has a deeper flow depth and lower flow intensity than the others in the series. The surface roller effect should result in a low predicted break point shear stress for experiment 11 (when compared with the shallower experiments 7, 8, 9 and 10) and the time series should be underpredicted. The lower flow intensity of experiment 11 would be expected to compound this effect.

However, the model overpredicts the time series for both experiments 4 and 11. Adding to the problem is the fact that experiments 7 and 8 have an even larger difference in flow aspect ratios than experiments 3 and 4 ($y_0/D = 1.3$ for experiment 7 and $y_0/D = 0.6$ for experiment 8), but the model time series prediction is excellent for both of these experiments. This suggests that either one or both of the overpredicted experiments (4 and 11) were flawed or that some interaction exists between the flow depth and velocity which has not been accounted for in the model.

Figures B-16 through B-30 show that the live bed scour model predictions are excellent in all cases. However, many of the live bed scour experiment reach equilibrium too quickly to judge the accuracy of model prediction early in the time series. The experiments plotted in Figures B-16 through B-19, B-23 through B-25, B-29 and B-30

have slower approaches to equilibrium. These experiments show good accuracy for the model throughout the time series'.

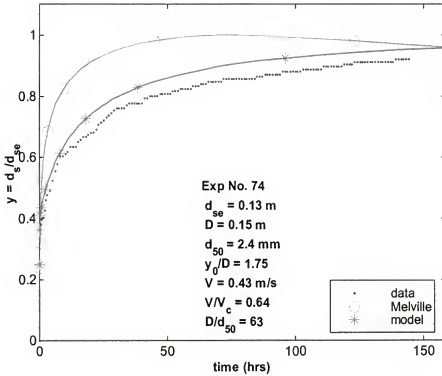
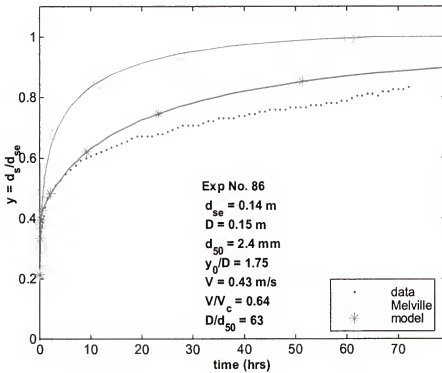
4.5 Testing the Model with Independent Researchers' Data

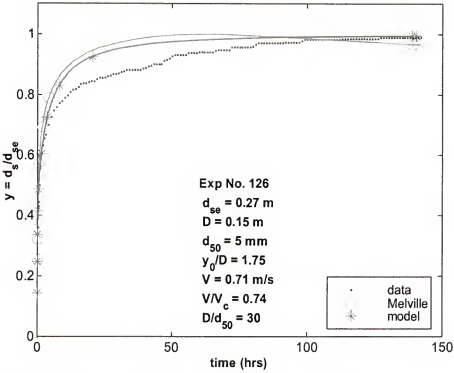
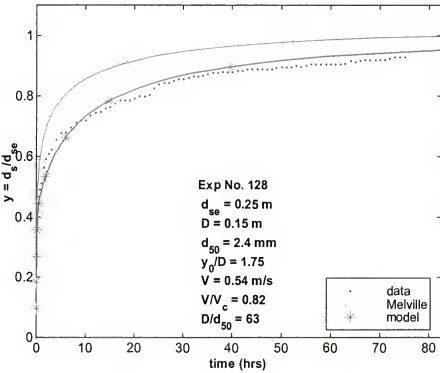
Mr. J. Sterling Jones, a Hydraulic Research Engineer at the Turner-Fairbanks Highway Research Center in McLean, Virginia, provided several clear water scour time series data sets from his experiments. The conditions for these data sets are contained in Table 4-4. Jones' experiment numbers are retained in the table.

Table 4-4. Jones data set flow and sediment parameters

Exp	D (m)	V (m/s)	d_{50} (mm)	V/V_c	y_0/D	D/d_{50}	d_{sc} (m)
74	0.152	0.43	2.4	0.64	1.75	63	0.13
86	0.152	0.43	2.4	0.64	1.75	63	0.14
126	0.152	0.71	5	0.74	1.75	30	0.27
128	0.152	0.54	2.4	0.82	1.75	63	0.25
134	0.152	0.42	1.2	0.93	1.75	127	0.24

The model presented in Eq. 4-27 was run for the parameters and conditions of Jones' experiments. Figures 4-51 through 4-55 are plots of the experimental and modeled scour depths (normalized with the equilibrium scour depth) versus time. The Melville and Chiew (1999) model results for these experiments are also plotted for comparison. Overall, the performance of the Eq. 4-27 model is good. Experiments 74 and 86 and portions of experiment 126 (Figures 4-51, 4-52 and 4-53), are overpredicted. However, this is conservative and from a design point of view, acceptable. This data set is for laboratory scale experiments (i.e., small D/d_{50}) and does not validate the model for large scale structures.

Figure 4-51. Jones Experiment 74: $V/V_c = 0.64$, $y_0/D = 1.75$, $D/d_{50} = 63$ Figure 4-52. Jones Experiment 86: $V/V_c = 0.64$, $y_0/D = 1.75$, $D/d_{50} = 63$

Figure 4-53. Jones Experiment 126: $V/V_c = 0.74$, $y_0/D = 1.75$, $D/d_{50} = 30$ Figure 4-54. Jones Experiment 128: $V/V_c = 0.82$, $y_0/D = 1.75$, $D/d_{50} = 63$

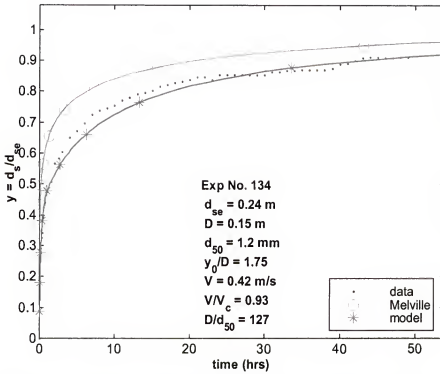


Figure 4-55. Jones Experiment 134: $V/V_c = 0.93$, $y_0/D = 1.75$, $D/d_{50} = 127$

CHAPTER 5 NUMERICAL STUDIES

5.1 Scour during a Storm Surge

A key application of the rate of local scour model (Chapter 4) is to provide design parameters for new and existing bridge piers that are subjected to high velocity, short duration flow events such as those produced by a hurricane storm surge. During such an event, structures (e.g., bridge piers) can experience velocities and water levels much higher than those seen under normal conditions. Present practice requires that the designer compute design scour depths based on the highest velocity anticipated during the event. Since the duration of many coastal flow events is short, on the order of hours, the design practice is thought to be overly conservative and costly. However, without a better understanding of the rate at which scour holes develop both the regulator and the practicing engineer must err on the conservative side.

For this study, five Florida bridges were examined: Fort George Bridge near Jacksonville, Ernest F. Lyons Bridge near Stuart, Jensen Beach Boulevard Bridge in Jensen Beach, the Bryant Patton Bridge across Apalachicola Bay to St. George Island and the Hathaway Bridge in Panama City. For each bridge location, a two-dimensional, depth averaged hydraulic model was developed by Ocean Engineering Associates, Inc. These models produced time histories of the flow velocity just upstream of the piers based on the water level input at the model boundaries. The water level was based on design storm conditions. Figures 5-1 and 5-2 show the velocity magnitude and water

depth hydrographs for typical piers at each bridge. The water depth in Figure 5-2 has been referenced to the low water level for each hydrograph.

With these hydrographs and the local sediment parameters, the rate of scour model developed in Chapter 4 can be applied in step fashion. That is, the unsteady storm surge velocity and depth hydrographs can be divided into time increments and each increment treated as a steady flow condition over that interval. Velocity magnitude, depth and equilibrium scour depth are averaged over the time increment and these values are used in the model. Totapally et al. (1999) indicated that the stepped hydrograph technique could be effectively used to approximate a storm surge event.

The procedure for calculating the local scour depth as a function of time, using the model developed here, is summarized below,

- Calculate V , u_* / u_{*c} , V / V_{cs} , y_0 / D , D / d_{s0} , d_{se} , the input sediment transport (for live bed conditions) and the transport coefficient (C) averaged over the time step.
- From these parameters, calculate the shear stress profile over the normalized scour depth ($y = d_s / d_{se}$).
- Based on the scour depth at the beginning of the time step, the d_{se} from step 1 and the shear stress profile from step 2, determine the shear stress to use in the scour model.
- Solve the model for this time step and increment the scour depth.
- Repeat time steps 1-4 starting with the previously computed scour depth and the flow conditions at the next time step.

The models of Melville and Chiew (1999) and Sumer et al. (1992) were also applied for comparison purposes (referred to as Melville and Sumer, respectively). Both of these models are empirical and based on exponential decay curves (Chapter 2).

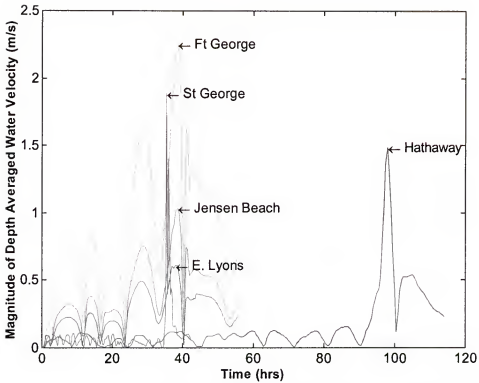


Figure 5-1. Velocity magnitude hydrographs for typical piers at each bridge examined

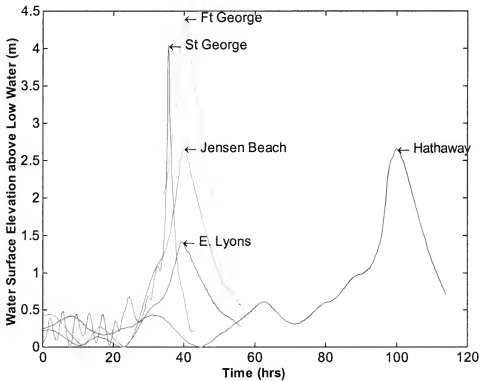


Figure 5-2. Water depth (above low water level) hydrographs for typical piers at each bridge examined

In each model, a time constant is first calculated and then applied to the exponential equation and the scour depth is calculated as a function of time. Inputs to the models include time, V/V_c , y_0/D , D/d_{50} and the equilibrium scour depth (d_{se}) for these conditions. Since the models are based on a steady condition, each time step must be treated as a separate steady state event. The input conditions are calculated as an average over each time step. Using the scour depth at the beginning of the time step, a value for the time corresponding to such a depth for the averaged conditions is calculated. Time is then incremented to the end of the step and the scour depth is computed as a function of time. This end condition for the time step is then used as the input condition for the next. The procedure is summarized below,

- Calculate V/V_c , y_0/D , D/d_{50} and d_{se} averaged over the time step.
- From these conditions, calculate the time constant for the equation.
- Using the parameters from steps 1 and 2, calculate the time at which the model would have reached the scour depth at the beginning of the time step.
- Increment this time by the given time step and calculate the scour depth at the end of the time step.
- Use this scour depth as the beginning scour depth for the next time step calculation.

The three models were applied to each pier on all of the five bridges and a time series of the scour depth as a function of time was obtained. The ranges of conditions for each location are tabulated in Table 5-1. Table 5-2 lists the ranges of the maximum percentage of the maximum equilibrium scour depth reached for each model during the design flow event. The percentage of the maximum equilibrium scour depth reached is the maximum scour depth reached divided by the equilibrium scour depth computed

using the maximum flow velocity achieved. That is, the design scour depth using the presently recommended approach.

Table 5-1. Range of conditions by location

Location	No. Piers	d_{50} (mm)	Range of Pier Diameters (m)	Range of D/d_{50}	Range of Velocity (m/s)	Range of Depth (m)
Hathaway	11	0.34	4.8 – 7.8	14065 – 22803	0.8 – 1.7	4.8 – 7.8
E. Lyons	28	0.17	2 – 6.7	11821 – 39187	0.6 – 1	2 – 6.7
Jensen Beach	9	0.20	2.5 – 4.2	12356 – 21066	0.7 – 1	2.5 – 4.2
Ft. George	15	0.20	0.8 – 2.5	3940 – 12708	2.1 – 2.7	0.8 – 2.5
St. George	161	0.20	1.9 – 8.2	9278 – 40773	1.7 – 2.8	1.9 – 8.2

Table 5-2. Percentage of Maximum Equilibrium Scour Depth Reached.

Location	UF Model		Melville		Sumer	
	Range	Average	Range	Average	Range	Average
Hathaway	11 – 26%	18%	99 – 100%	99%	29 – 77%	55%
E. Lyons	10 – 78%	33%	99 – 100%	100%	31 – 96%	70%
Jensen Beach	29 – 59%	46%	99 – 100%	100%	55 – 90%	78%
Ft. George	92 – 99%	97%	100%	100%	100%	100%
St. George	3 – 70%	24%	91 – 100%	96%	47 – 100%	85%

Figures 5-3 through 5-7 are plots of the scour depth as a function of time for a typical pier at each location. The model presented in this dissertation is labeled UF in the figures. The figures and Table 5-2 indicate that the Melville model quickly approaches the transient equilibrium at each time step, achieving almost 100% of the maximum transient equilibrium scour depth, d_{se-max} . On the other hand, the model presented here and that of Sumer predict a range of percentages of maximum equilibrium scour depth (from as little as 10% up to 100%).

Table 5-2 shows that all the model predictions for the Ft. George piers approach d_{se-max} , with both the Melville and Sumer models reaching the maximum value. Figure 5-6 shows the model presented here, which accounts for upstream bed-load sediment entering the scour hole, predicting a period of infilling following the peak scour depth on one of the Ft. George piers. Such infilling is expected under transient conditions,

however, neither the Melville nor the Sumer model make use of a sediment input component and therefore cannot predict infilling of the scour hole.

Table 5-1 indicates that the Ft. George velocities are on average the highest of the five locations and the piers are the smallest. Recalling from Chapter 3, section 3.4.1 that high live-bed V/V_c and low D/d_{50} both result in a shortened time to equilibrium it is not surprising that the d_{sc-max} is reached for the piers at this location. It is also interesting to note that the time at which the maximum is reached in Figure 5-6 is nearly the same for all three models.

For the UF model and the Sumer model, the lowest average percentage occurs at the Hathaway piers. Figure 5-1 shows that the Hathaway velocities are low for most of the period studied. Up to approximately 90 hours V/V_c is actually below the minimum for local scour. When the storm surge does strike, high velocities are only experienced for a very short period, approximately 20 hours. The extremely limited duration is the reason for the low percentage of d_{sc-max} predicted. For the Melville model, the time to equilibrium is so short that the duration is of little importance.

The next lowest percentage occurs at the St. George piers. Again, Figure 5-1 indicates this location has a very short period of high velocities. The period is actually shorter than the Hathaway surge, however the velocities are higher. In this case the effects of the higher velocity outweigh the effects of the short duration to produce near maximum scour depths during the design flow event. On the other hand, the range of percentages is much greater at the St. George Island Bridge piers than at any other location, 3 to 70% for the model presented here. The likely cause of this is the wide range of pier sizes. St. George D/d_{50} ranges from 9000 to 40000 (the greatest range of all

the locations). The effect of this D/d_{50} range is compounded by a large velocity range a (1.7 to 2.8 m/s). With both of these effects, the wide range of percentages would be expected.

Finally, the Jensen Beach and Ernest Lyons piers have very similar characteristics and similar responses. Figure 5-1 shows that neither location is subjected to extraordinarily high velocities (their maximum velocity is 1 m/s). However, they are both subject to relatively high velocities for long durations (on the order of 20 to 30 hours). Figures 5-4 and 5-5 show the scour depth increasing steadily for both this model and the Sumer model, reaching a plateau of approximately 50% of d_{sc-max} just as the velocities in Figure 5-1 are dropping off. If continued further, the model would likely show that 50% of d_{sc-max} will be reached and maintained as V/V_c drops below the minimum for scouring.

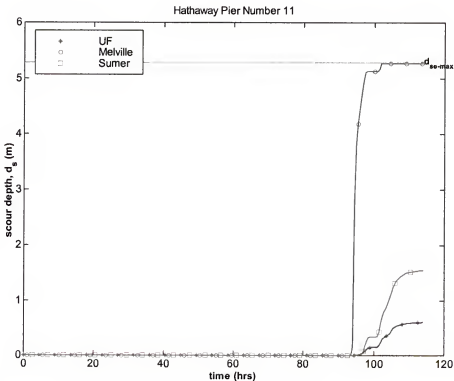


Figure 5-3. Scour depth as a function of time for Hathaway

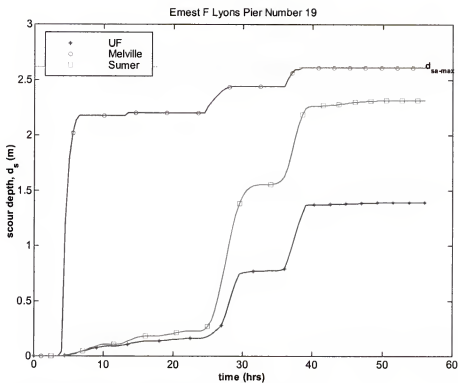


Figure 5-4. Scour depth as a function of time for Lyons

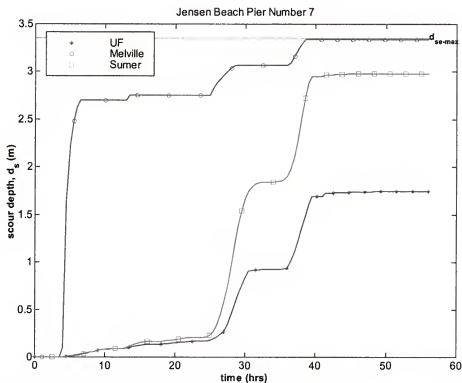


Figure 5-5. Scour depth as a function of time for Jensen Beach

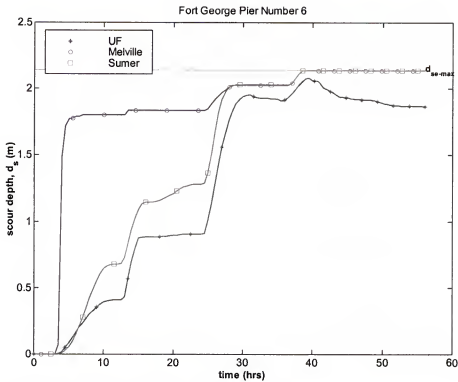


Figure 5-6. Scour depth as a function of time for Ft. George

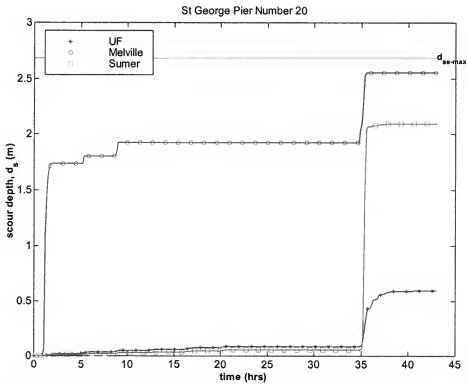


Figure 5-7. Scour depth as a function of time for St. George

5.2 Scour due to Tides

A second type of unsteady flow common to coastal areas is that due to astronomical tides. Many bridges located at or near tidal inlets experience the reversing flows created by the periodic variation in the water surface on the open coast.

Walker (1995) conducted a field study to measure the local pier scour due to tidal flows by filling a pre-existing scour hole around a bridge pile in Destin, Florida and monitoring the subsequent scour process. After filling the hole from a nearby borrow area with similar sand, he monitored the rate at which the tidal flows restored the scour hole to its initial condition. Walker found that the scour hole was restored to its original depth in approximately 170 hours. In that time the tide scoured approximately 70 cm of fill from the hole. Walker chose a 24 inch wide square pile with beveled edges and 3 to 4 inches of growth on all sides for a total width of approximately 30 inches (0.76 m). The sediment of both the scour hole and the borrow site had a median diameter of 0.28 mm. The mean water depth was approximately 3 m and the tide was diurnal with a range of approximately 1 m. Maximum tidal velocities ranged from 0.4 to 0.5 m/s. Figure 5-8 is a plot of water depth and flow intensity for a similar tide to that measured by Walker with V_c calculated for sediment with a d_{50} of 0.28 mm.

The tide in Figure 5-8 and the pile and sediment parameters described above were used as input to the model. To account for the fact that the Destin pile was square and the model was developed for a circular pile, a shape factor of 1.15 was applied to the model. This increased the model pile diameter from 0.76 m to 0.9 m.

Figure 5-9 plots the model results along with data from Walker (1995). In the figure, the d_{sc-max} line is the equilibrium scour depth calculated at the maximum V/V_c

from Figure 5-8(b). The d_{se-rms} line is the equilibrium scour depth calculated for V_{rms}/V_c where V_{rms} is the root mean square value of the magnitude of the depth averaged velocity. Agreement between the model and the data is fair, though the model does under-predict the final scour depth measured by Walker.

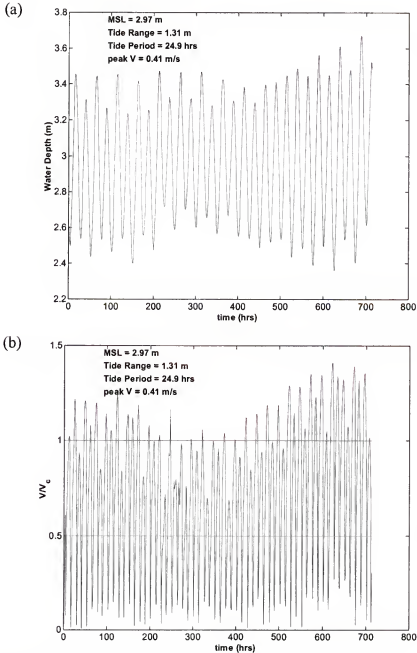


Figure 5-8. Diurnal tide (a) water depth and (b) V/V_c as functions of time, MSL = 2.97 m, Tide Range = 1.3 m, Tide Period = 24.9 hrs, peak Velocity = 0.4 m/s

A number of conditions differ between the Walker study and the model input.

The tides are not identical. The fill material was loose and not compacted as it had been before each of the experiments of Chapter 3. Either of these facts could explain the difference between the model results and the data. However, despite the differences, the similarity between the model results and the data do indicate that, once calibrated, the model can potentially provide the time rate of scour information needed by the designer.

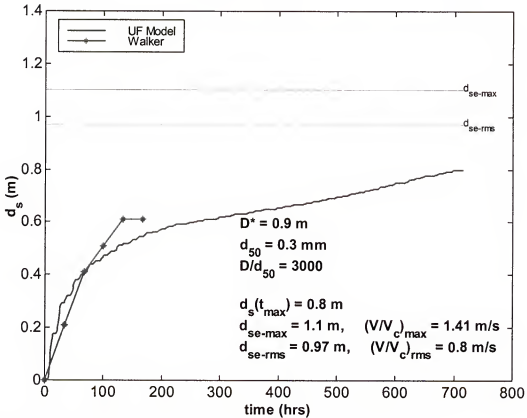


Figure 5-9. Predicted scour depth for the diurnal tide study, $D = 0.9 \text{ m}$

To examine the effect of varying the tidal period, the diurnal tide in Figure 5-8 was modified by halving its time steps to construct a semi-diurnal tide. Tidal range, velocities magnitude and mean sea level were not changed. This tide was input to the UF model and the model was run for a cylinder diameter of 0.75 m. The result is plotted along with the diurnal tide result in Figure 5-10. There is little difference between the

two until approximately 300 hours when the semi-diurnal model rises quickly to what appears to be an equilibrium state.

By repeating the 700 hour semi-diurnal tidal record three times the record was increased to 2100 hours. Similarly, the diurnal tidal record was increased to the same time and both were inputted to the model to investigate the effects beyond the 700 hours of Figure 5-10. Figure 5-11 shows the results. At roughly 500 hours the diurnal model undergoes an increase similar to that of the semi-diurnal at 300 hours. Thereafter, the two plots again merge and there is little difference.

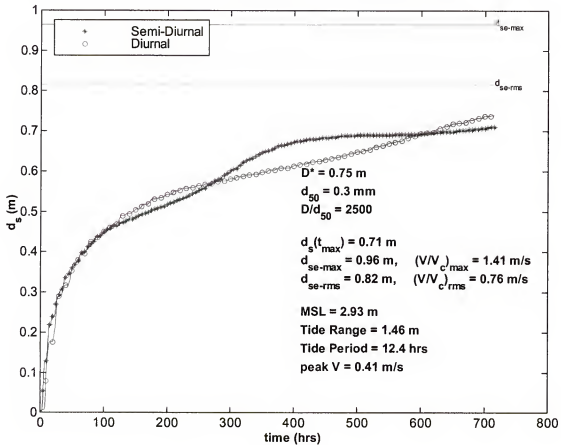


Figure 5-10. Diurnal and semi-diurnal tide effects comparison

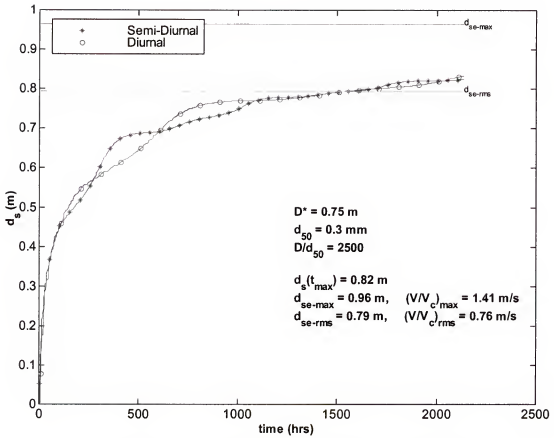


Figure 5-11. Extended time comparison of diurnal and semi-diurnal tide effects

CHAPTER 6 SUMMARY AND CONCLUSIONS

6.1 Summary

A model has been developed to describe the growth of a scour hole about a single, circular cylinder as a function of time. This model is based on the geometry of the scour hole and the physics of the fluid flow and sediment transport into and out of the hole and around the cylinder. Like all mathematical models of this nature the geometries and processes have been simplified to make the problem tractable. The intent is to include the primary forces/processes in the model so that it is a reasonable representation of the actual situation. The following assumptions were made in the development of the model:

- The sediment is non-cohesive and uniform in size.
- The scour hole is symmetric and may be approximated by the frustum of an inverted right circular cone.
- The shape of the scour hole does not change as the hole develops and the sides remain at the submerged angle of repose of the sediment. The uniform and symmetric shape of the scour hole is maintained by sediment sliding or avalanching down the sides of the scour hole into the region of scour near the structure.
- Erosion in the scour hole is limited to a small area at the bottom of the hole and immediately adjacent to the cylinder. Sediment is transported out of the region of scour by the high bed shear stresses in this area and transported out of the scour hole by the mean flow.

- The size of the erosion zone does not change with scour depth and its width is only a function of cylinder diameter.
- Only bed-load sediment transport from upstream of the scour hole is considered in the sediment budget for the hole. Suspended sediment entering the hole is assumed to remain in suspension and to be transported out of the hole without impacting the local scour processes.
- The sediment transport associated with local scour can be approximated by using a bed-load transport formula with an effective shear stress that represents a time average of the complex, unsteady forces exerted on the bed in this region.
- The transport coefficient for this transport formula is constant throughout the scour development.
- The effective shear stress as a function of normalized scour depth (d_s/d_{se}) used in the transport formula may be accurately modeled by simple geometric shapes (parabolas and straight lines) that can be determined from the structure, flow and sediment parameters.

6.2 Conclusions and Further Work

The assumptions in section 6.1 form the basis of the rate of local scour model developed in Chapter 4. The features of the effective shear stress versus normalized scour depth profile have been related to known flow, sediment and structure parameters for a given situation. The turbulent flow structures (secondary flows) associated with the structure-induced scour problem include:

- the horseshoe vortex,
- the surface roller or bow wave,

- the downflow,
- the wake vortices, and
- turbulence structures developed at the separation point at the leading edge of the scour hole.

These flow structures are only addressed indirectly in the model through the use of nondimensional groups believed to characterize these structures. Improving the current theory for characterizing these flow structures as they relate to the scour process would enhance the level of confidence when using of the model beyond the bounds of the existing data.

Chapter 5 showed the value of being able to predict the maximum scour depth that will be reached during a particular transient event (e.g., a storm surge) or due to regular periodic events (e.g., tidal flows). Optimizing the design of a coastal structure requires the consideration of many complex interactions: flow event duration, structure size, maximum flow velocity and water level reached during the event. Though not examined here, it is also possible to use the model to account for variations in bed material properties with depth by incorporating different sediment sizes at different elevations.

The effect of large values of D/d_{50} on the initial transport rate and so the transport coefficient for live bed scour conditions is still undetermined. The few live bed scour experiments with large values of D/d_{50} also have flow intensities that place them at the point of convergence of the curves in Figure 4-24. While it is reasonable that the value of the transport coefficient becomes independent of D/d_{50} as D/d_{50} becomes large as shown in Figure 4-25, more data is needed before definite conclusions can be drawn.

Similarly, the effect of large values of D/d_{50} on the initial transport rate under clear water scour conditions is still poorly understood. The curve in Figure 4-28 shows that the initial transport rate becomes independent of D/d_{50} as D/d_{50} becomes large. There is an indication that the initial transport (for clear water scour) increases slowly with increasing D/d_{50} (for $D/d_{50} > 1200$) in Figure 4-28. However, this trend is based on only one experiment and more data is needed before definite conclusions can be drawn.

The modeling of experiments 4 and 11 indicate that there are opposing effects of flow velocity and flow depth on the break point in shear stress curve under clear water scour conditions.

Equation 4-35 suggests that the depth at which the shear stress reaches its peak is limited to $0.68d_{se}$. However, it is conceivable that under some live bed conditions the effective shear stress might simply increase until the equilibrium depth is reached. The effect of increasing equilibrium scour depth with increasing V/V_c may be the cause of this limit, but confirmation will require additional evidence.

A division in the live bed scour flow intensity range was observed in Figure 4-30. For flow intensities values above 2, the peak depth appears independent of D/d_{50} and only dependent on V/V_c . For flow intensities values below 2, the peak depth depends on both of these parameters. However, there was no experimental data for D/d_{50} greater than 600 with $V/V_c > 2$. Experiments with D/d_{50} greater than 1000 may show additional dependencies. Nonetheless, a division does seem to exist and has not been explained.

Figures 6-1 and 6-2 show the parameter ranges (shaded areas) for which the scour and effective shear stress models have been validated in this dissertation. In these ranges the models may be applied with confidence. The hashed area in Figure 6-2 is an area for

which there is little data; however the surface roller effect of y_0/D should not affect the model in this range. Therefore, the model may be applied in this area with only slightly reduced confidence.

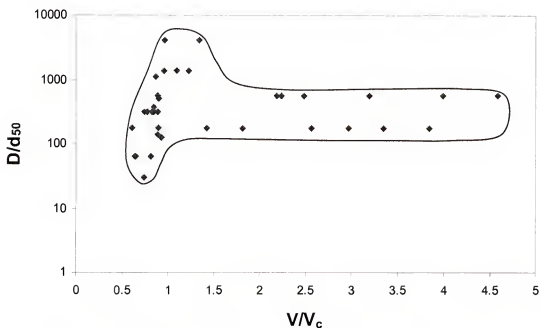


Figure 6-1. Range of D/d_{50} vs. V/V_c parameters covered by experimental data used in this analysis

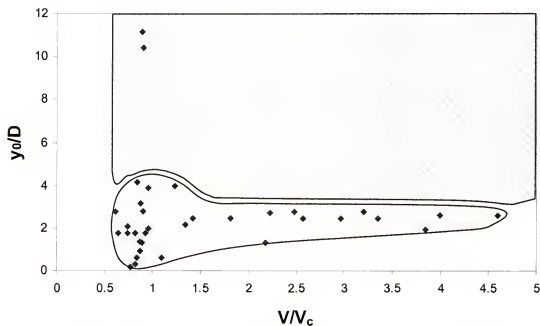


Figure 6-2. Range of y_0/D vs. V/V_c parameters covered by experimental data used in this analysis

The piecewise approximation of the effective shear stress as a function of normalized scour depth greatly simplifies the shear stress profile by using basic geometric shapes (parabolas and lines). A better approximation that incorporates a smoother change in slope at the break point may be possible.

It may be possible to adapt this model to an asymmetric scour hole. The downstream portion of the scour hole generally has a gentler slope with a mound of sediment. Modifying the initial geometry may account for this characteristic.

Finally, the equilibrium scour depth must be known or calculated prior to modeling the shear stress profile. If it is possible to model the shear stress profile using a different parameter to normalize the depth, e.g. the cylinder diameter, this model might be able to predict the equilibrium scour depth itself.

The model presented here for the development of the scour hole with time has the potential to vastly improve the designer's ability to predict the effects of unsteady flow events. This would allow better optimization of the design of many coastal structures and potentially reduce the costs of foundations. However, additional work is needed to validate and/or improve the model for larger structures in high velocity flows. At the moment, the model can be a useful tool in estimating rates of scour under steady and unsteady flow conditions.

APPENDIX A DATA PLOTS AND FIT CURVES

This appendix contains plots of the time series data for the scour experiments and fit curves for the data. Each plot shows the scour hole depth in meters as a function of time in hours. Each figure contains two plots. The first plot is of the full data set. The second plot shows the first few hours of each experiment and the corresponding fit. Chapter 3 contains the details of the fit curve development.

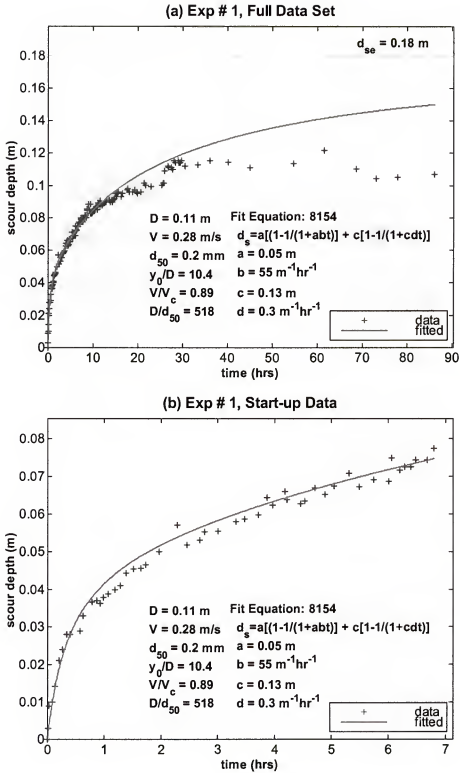


Figure A-1. UF Experiment 1: $V/V_c = 0.9$, $y_0/D = 10.4$, $D/d_{50} = 518$

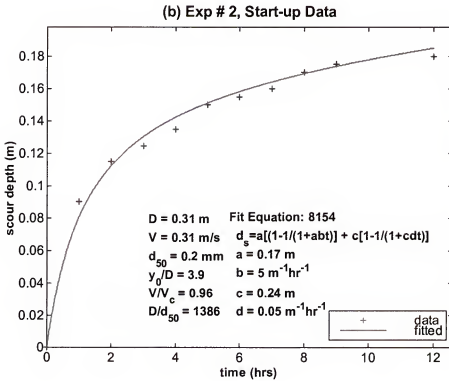
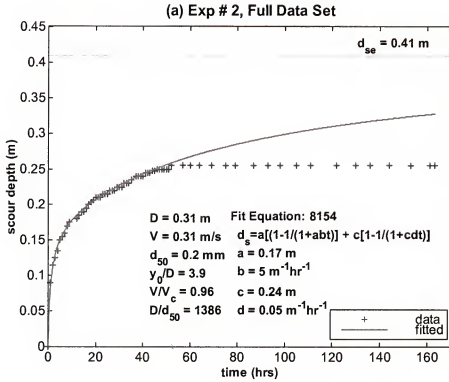


Figure A-2. UF Experiment 2: $V/V_c = 0.96$, $y_0/D = 3.9$, $D/d_{50} = 1386$

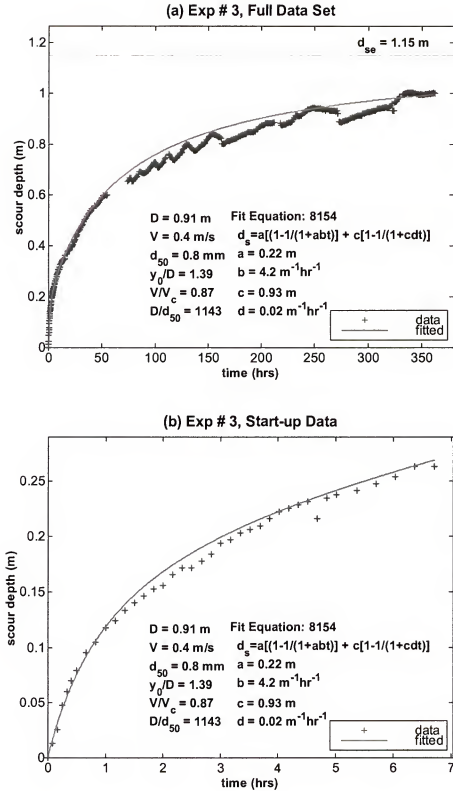
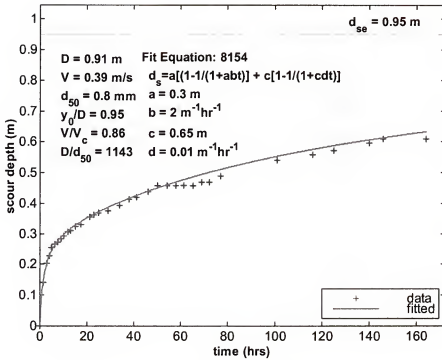
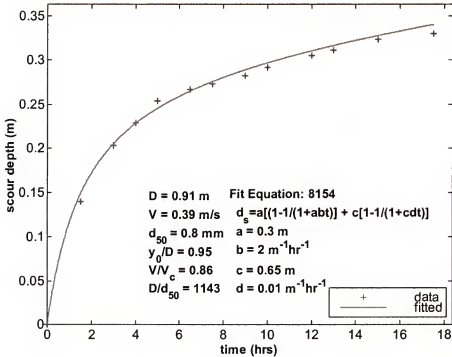


Figure A-3. UF Experiment 3: $V/V_c = 0.87$, $y_0/D = 1.39$, $D/d_{50} = 1143$

(a) Exp # 4, Full Data Set



(b) Exp # 4, Start-up Data

Figure A-4. UF Experiment 4: $V/V_c = 0.86$, $y_0/D = 0.95$, $D/d_{50} = 1143$

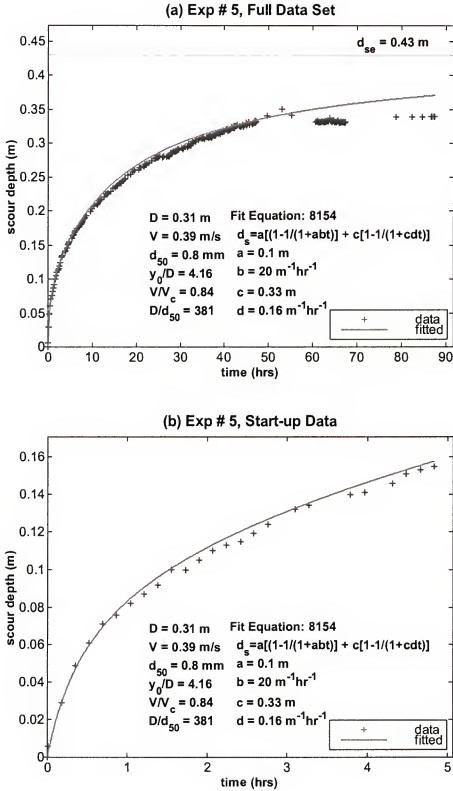


Figure A-5. UF Experiment 5: $V/V_c = 0.84$, $y_0/D = 4.16$, $D/d_{50} = 1143$

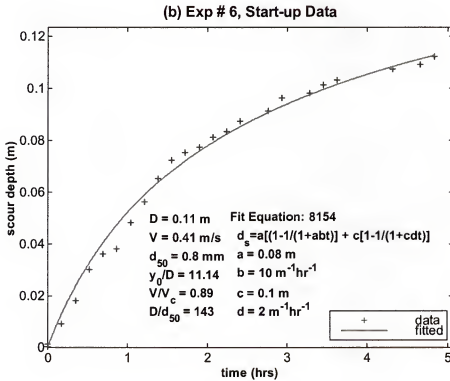
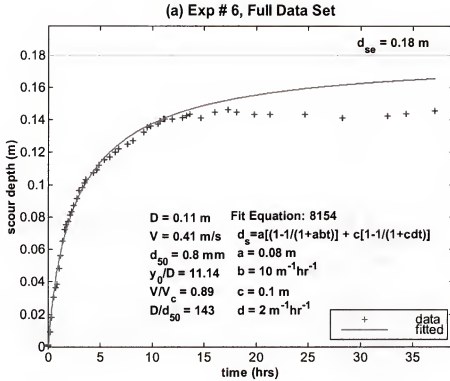


Figure A-6. UF Experiment 6: $V/V_c = 0.89$, $y_0/D = 11.14$, $D/d_{50} = 143$

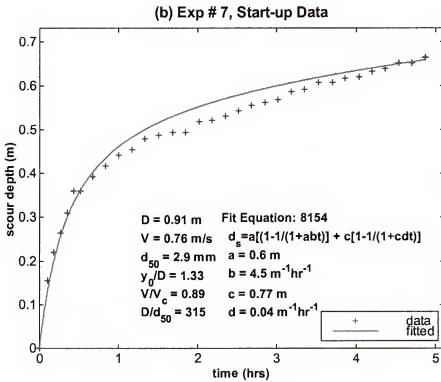
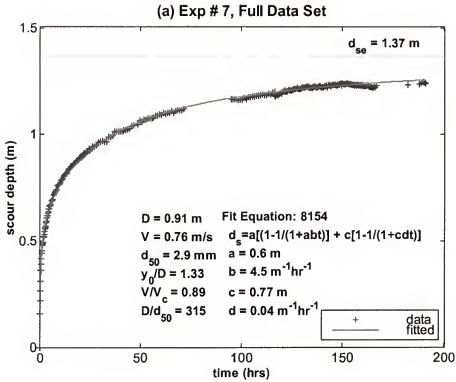
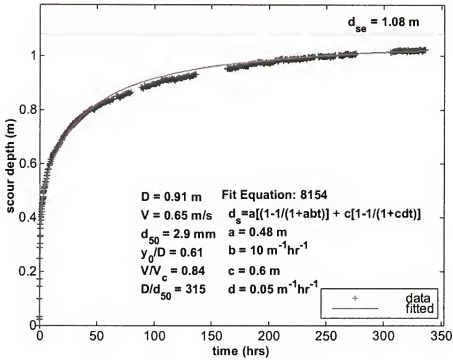
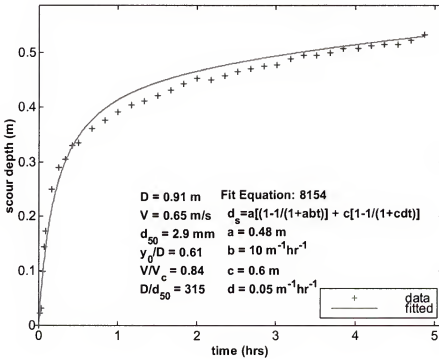


Figure A-7. UF Experiment 7: $V/V_c = 0.89$, $y_0/D = 1.33$, $D/d_{50} = 315$

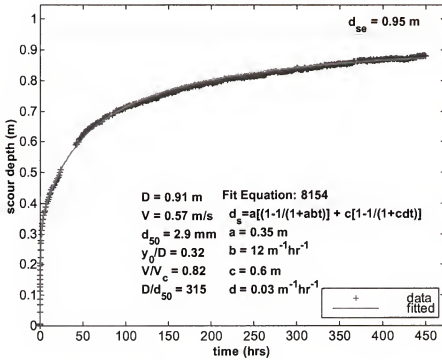
(a) Exp # 8, Full Data Set



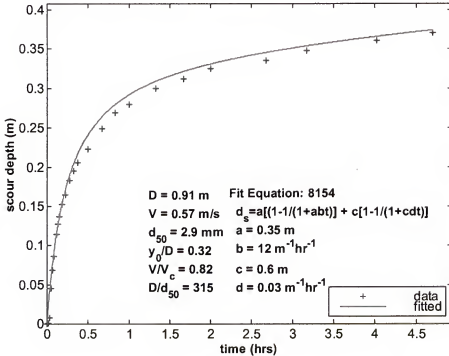
(b) Exp # 8, Start-up Data

Figure A-8. UF Experiment 8: $V/V_c = 0.84$, $y_0/D = 0.61$, $D/d_{50} = 315$

(a) Exp # 9, Full Data Set



(b) Exp # 9, Start-up Data

Figure A-9. UF Experiment 9: $V/V_c = 0.82$, $y_0/D = 0.32$, $D/d_{50} = 315$

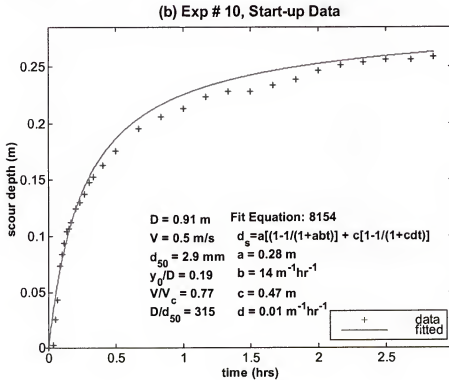
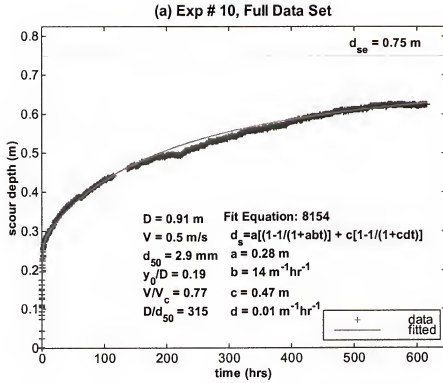


Figure A-10. UF Experiment 10: $V/V_c = 0.77$, $y_0/D = 0.19$, $D/d_{50} = 315$

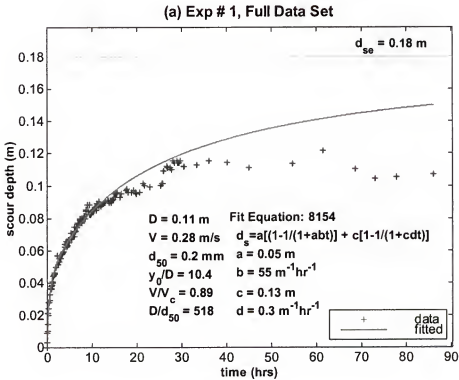
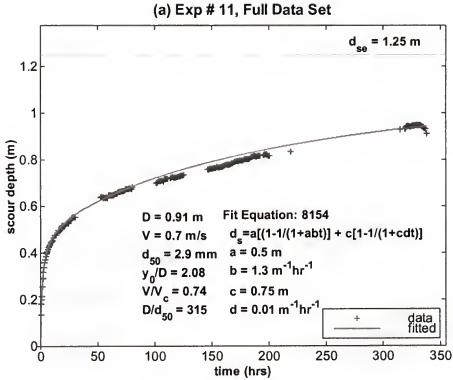


Figure A-11. UF Experiment 11: $V/V_c = 0.74$, $y_0/D = 2.08$, $D/d_{50} = 315$

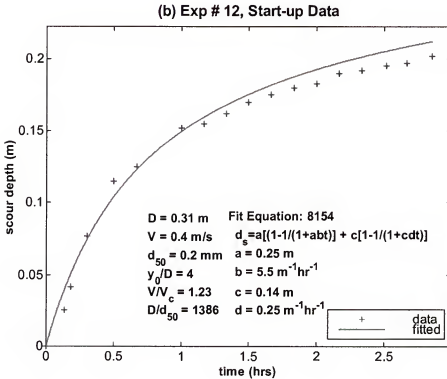
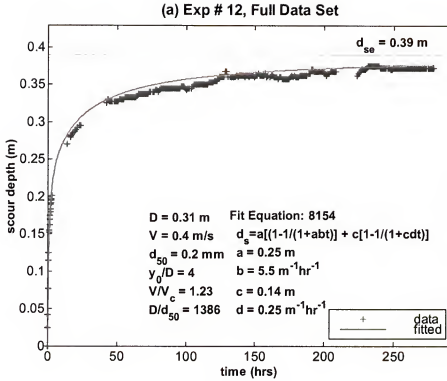
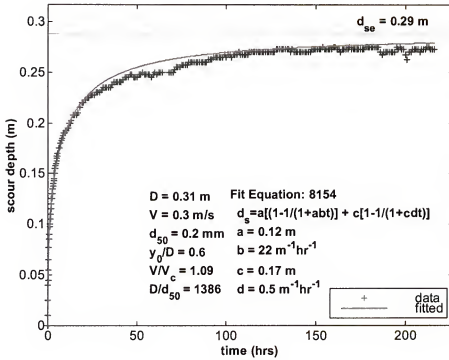
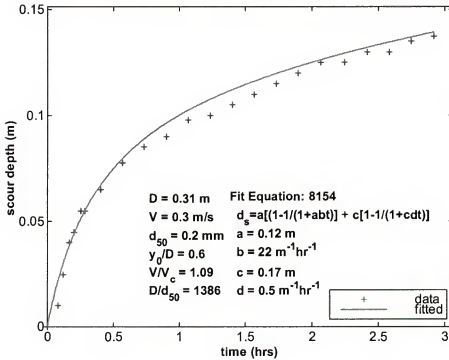


Figure A-12. UF Experiment 12: $V/V_c = 1.23$, $y_0/D = 4$, $D/d_{50} = 1386$

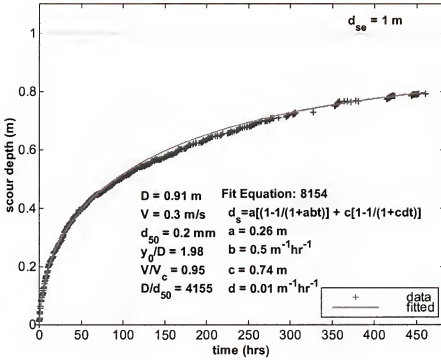
(a) Exp # 13, Full Data Set



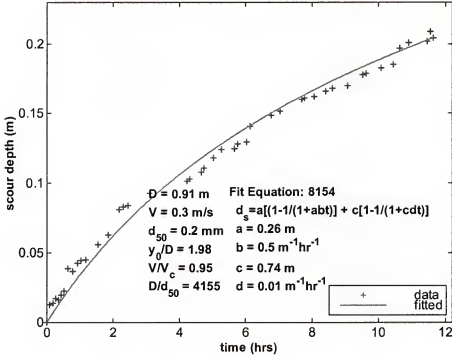
(b) Exp # 13, Start-up Data

Figure A-13. UF Experiment 13: $V/V_c = 1.1$, $y_0/D = 0.6$, $D/d_{50} = 1386$

(a) Exp # 14, Full Data Set



(b) Exp # 14, Start-up Data

Figure A-14. UF Experiment 14: $V/V_c = 0.95$, $y_0/D = 1.98$, $D/d_{50} = 4155$

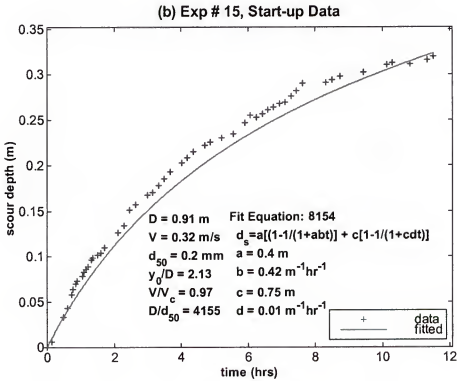
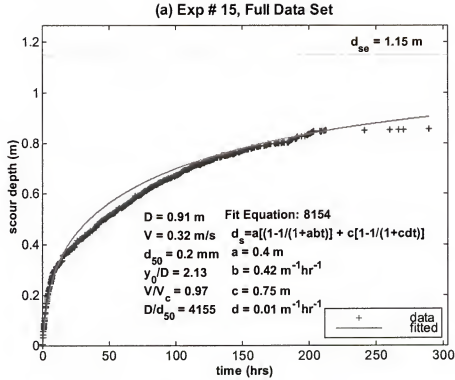
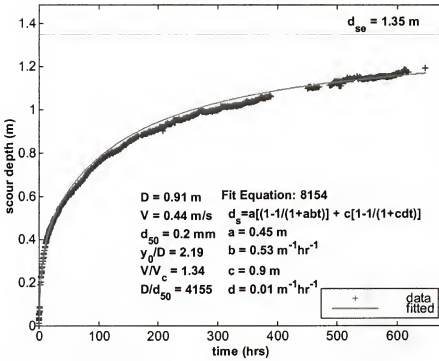
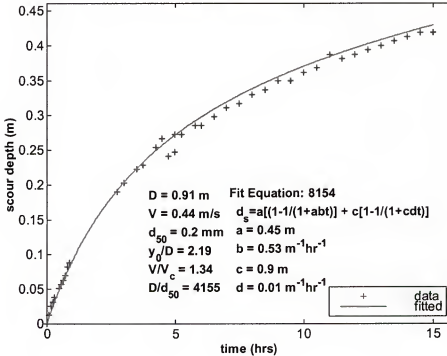


Figure A-15. UF Experiment 15: $V/V_c = 0.97$, $y_0/D = 2.13$, $D/d_{50} = 4155$

(a) Exp # 16, Full Data Set



(b) Exp # 16, Start-up Data

Figure A-16. UF Experiment 16: $V/V_c = 1.34$, $y_0/D = 2.19$, $D/d_{50} = 4155$

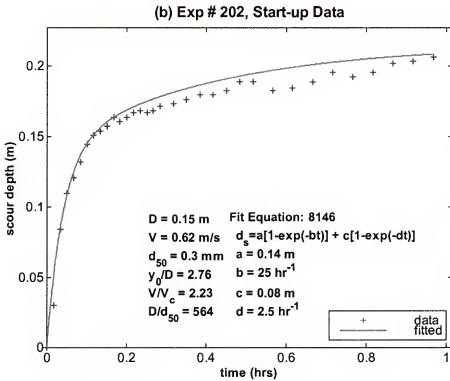
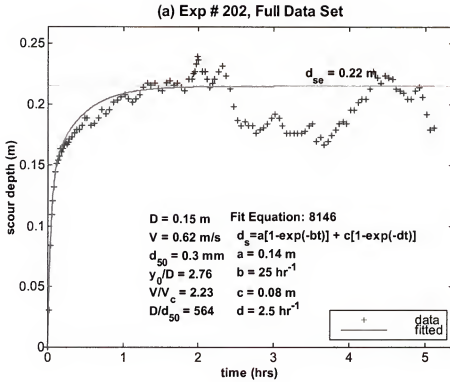


Figure A-17. UF Experiment 202: $V/V_c = 2.23$, $y_0/D = 2.76$, $D/d_{50} = 564$

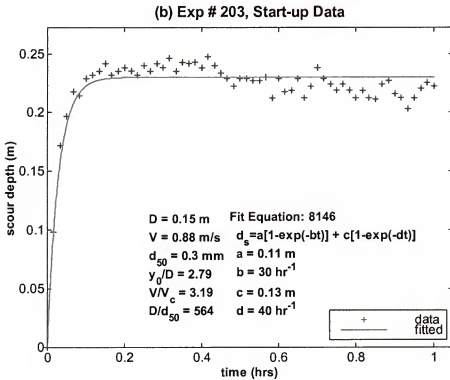
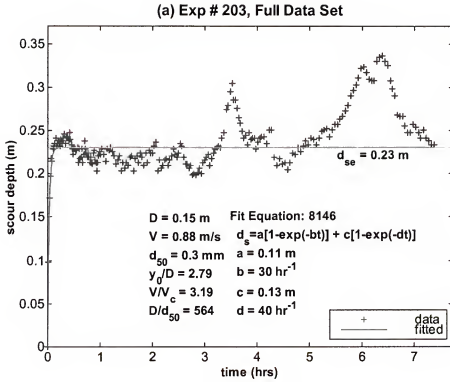


Figure A-18. UF Experiment 203: $V/V_c = 3.2$, $y_0/D = 2.79$, $D/d_{50} = 564$

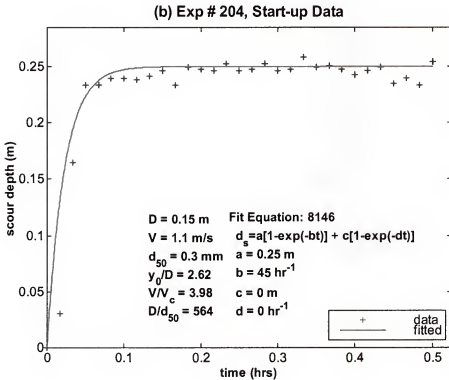
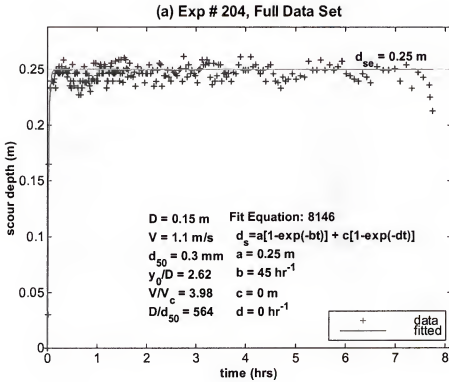


Figure A-19. UF Experiment 204: $V/V_c = 3.99$, $y_0/D = 2.62$, $D/d_{50} = 564$

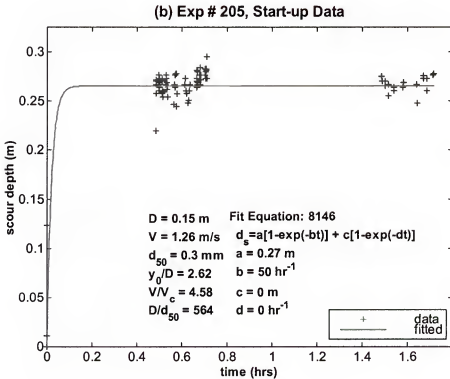
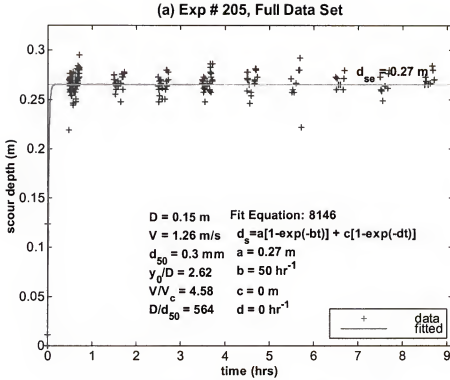


Figure A-20. UF Experiment 205: $V/V_c = 4.59$, $y_0/D = 2.62$, $D/d_{50} = 564$

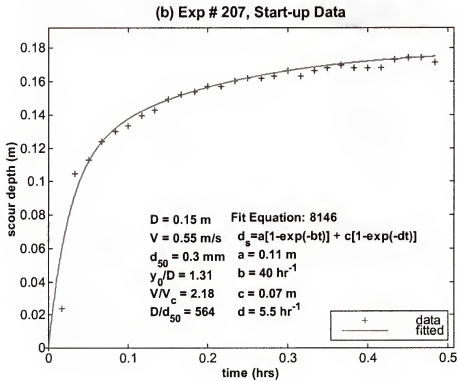
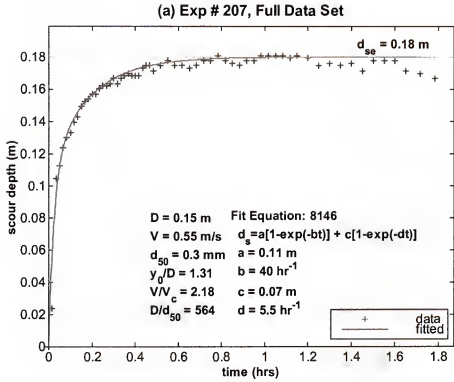


Figure A-21. UF Experiment 207: $V/V_c = 2.18$, $y_0/D = 1.31$, $D/d_{50} = 564$

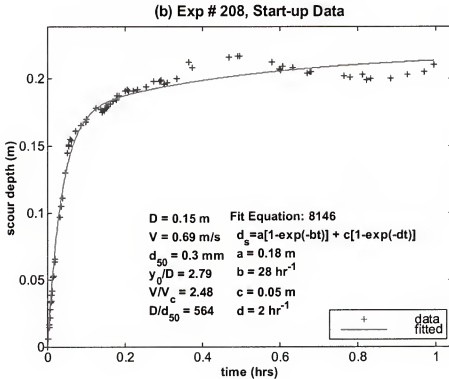
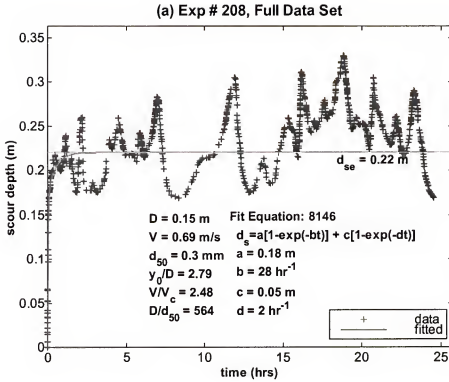


Figure A-22. UF Experiment 208: $V/V_c = 2.49$, $y_0/D = 2.79$, $D/d_{50} = 564$

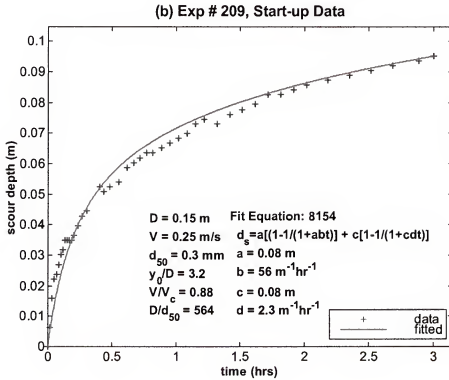
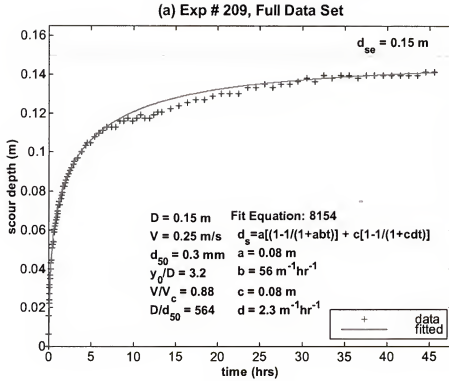


Figure A-23. UF Experiment 209: $V/V_c = 0.88$, $y_0/D = 3.2$, $D/d_{50} = 564$

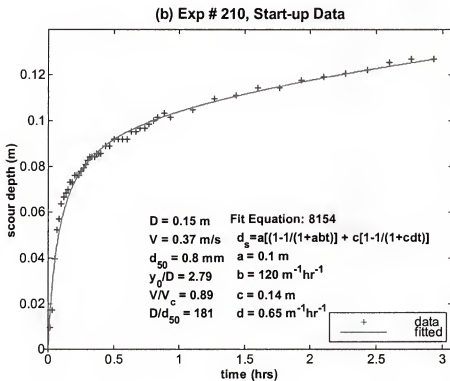
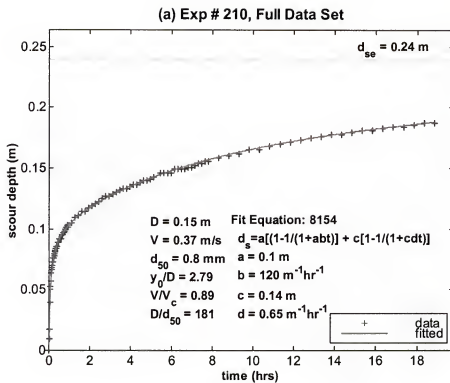


Figure A-24. UF Experiment 210: $V/V_c = 0.89$, $y_0/D = 2.79$, $D/d_{50} = 181$

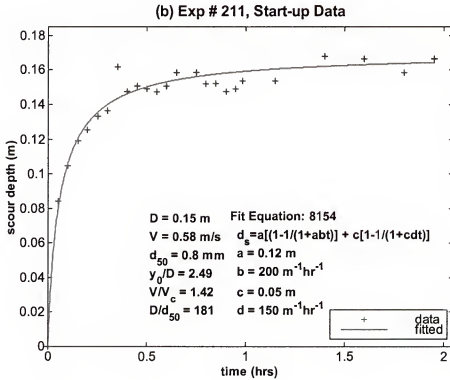
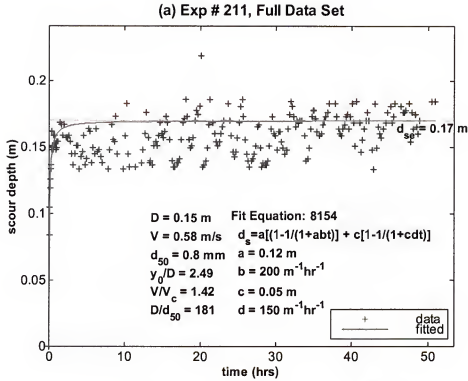


Figure A-25. UF Experiment 211: $V/V_c = 1.42$, $y_0/D = 2.49$, $D/d_{50} = 181$

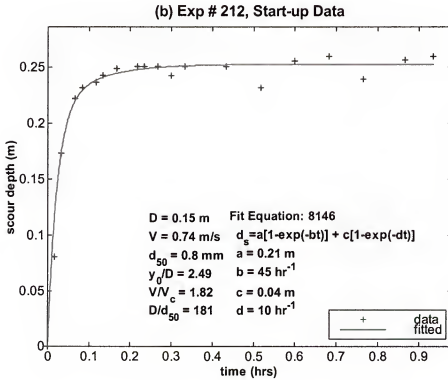
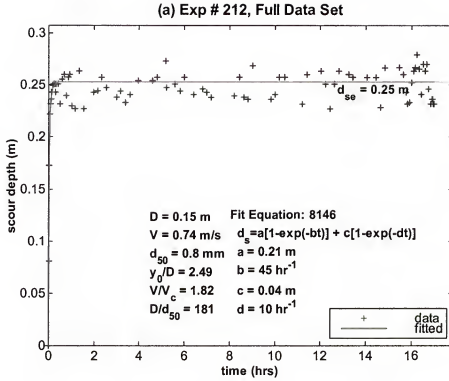


Figure A-26. UF Experiment 212: $V/V_c = 1.81$, $y_0/D = 2.49$, $D/d_{50} = 181$

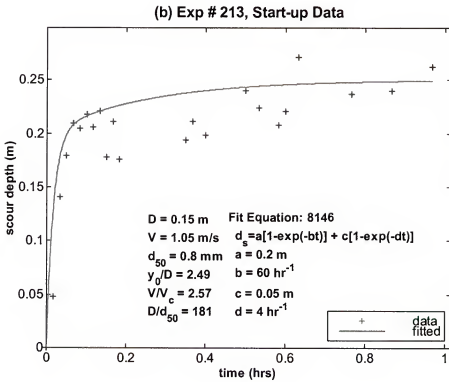
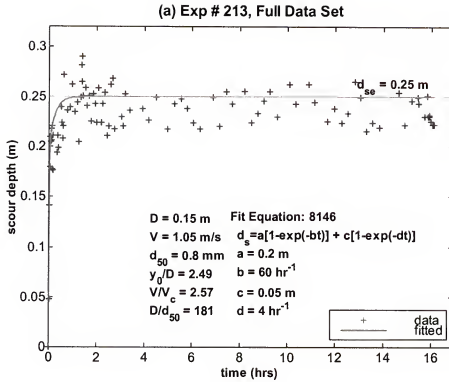


Figure A-27. UF Experiment 213: $V/V_c = 2.57$, $y_0/D = 2.49$, $D/d_{50} = 181$

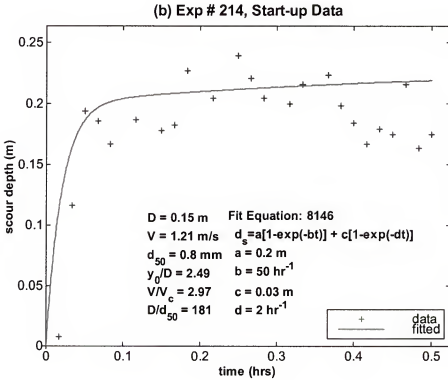
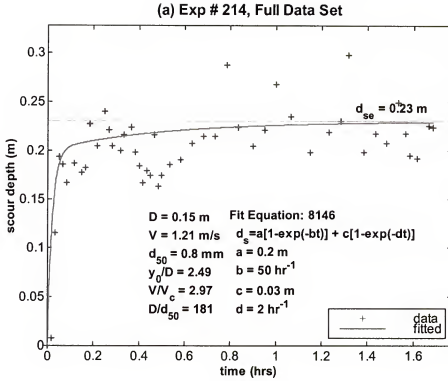
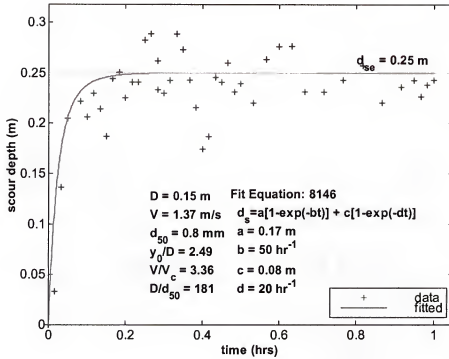
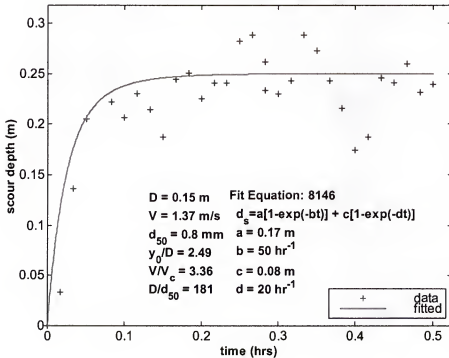


Figure A-28. UF Experiment 214: $V/V_c = 2.97$, $y_0/D = 2.49$, $D/d_{50} = 181$

(a) Exp # 215, Full Data Set



(b) Exp # 215, Start-up Data

Figure A-29. UF Experiment 215: $V/V_c = 3.36$, $y_0/D = 2.49$, $D/d_{50} = 181$

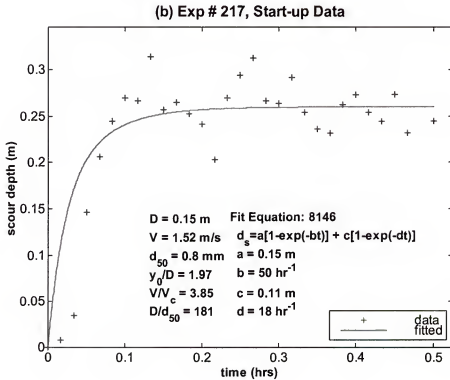
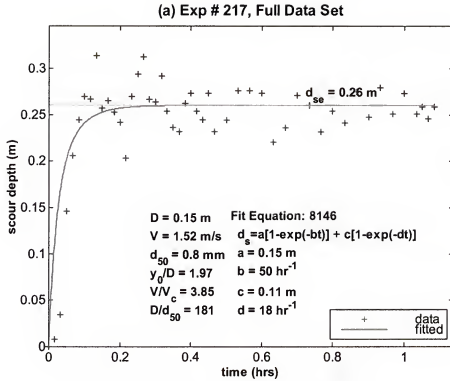


Figure A-30. UF Experiment 217: $V/V_c = 3.85$, $y_0/D = 1.97$, $D/d_{50} = 181$

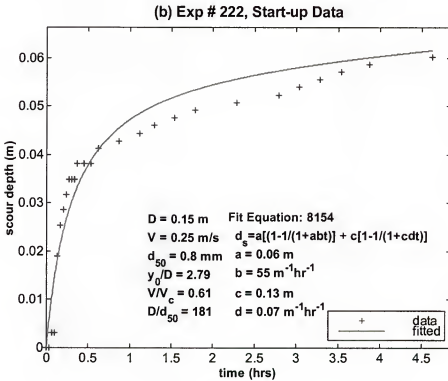
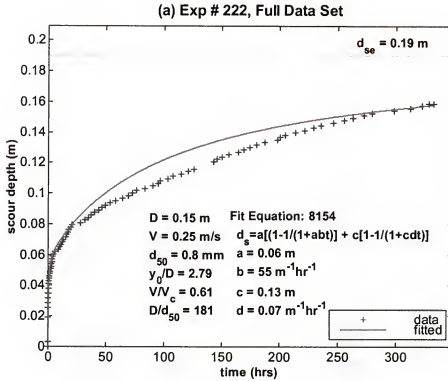


Figure A-31. UF Experiment 222: $V/V_c = 0.61$, $y_0/D = 2.79$, $D/d_{50} = 181$

APPENDIX B MODEL PLOTS

This appendix compares plots of the time series data for the scour experiments and the model calculations described in Chapter 4. Each plot shows the scour hole depth normalized with the equilibrium scour depth as a function of time in hours. In each plot the curve designated Melville was developed by the Melville and Chiew (1999) time series calculation described in Chapter 2. The curve designated model is the time series developed from Chapter 4.

B.1 Clear Water Scour Experiment Model Plots

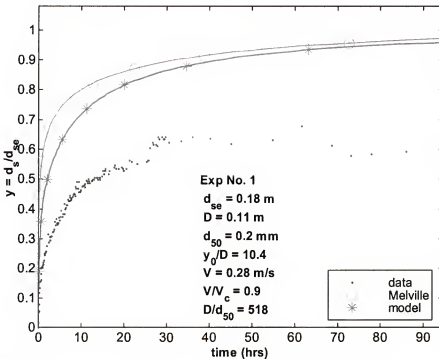


Figure B-1. UF Experiment 1: $V/V_c = 0.9$, $y_0/D = 10.4$, $D/d_{50} = 518$

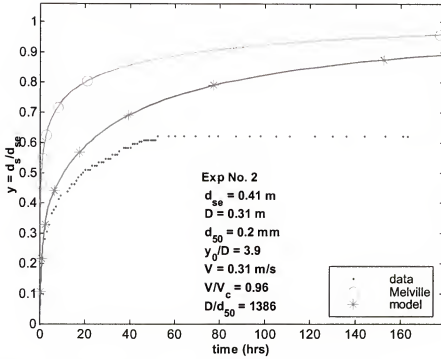


Figure B-2. UF Experiment 2: $V/V_c = 0.96$, $y_0/D = 3.9$, $D/d_{50} = 1386$

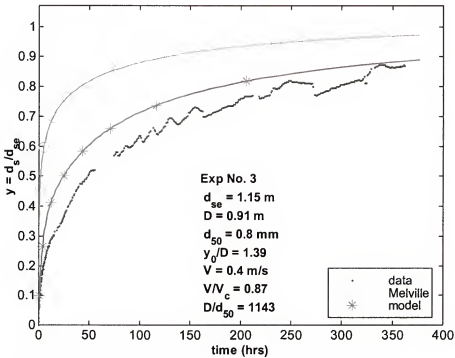


Figure B-3. UF Experiment 3: $V/V_c = 0.87$, $y_0/D = 1.39$, $D/d_{50} = 1143$

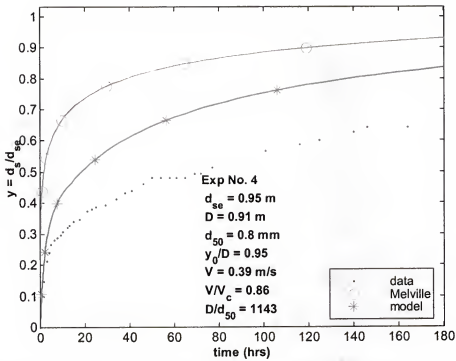


Figure B-4. UF Experiment 4: $V/V_c = 0.86$, $y_0/D = 0.95$, $D/d_{50} = 1143$

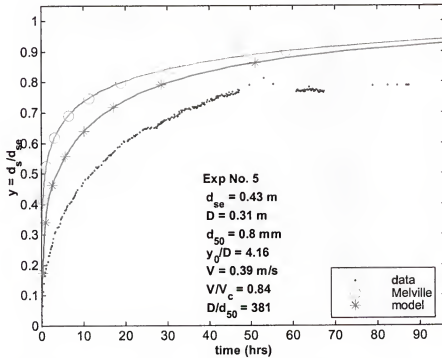


Figure B-5. UF Experiment 5: $V/V_c = 0.84$, $y_0/D = 4.16$, $D/d_{50} = 1143$

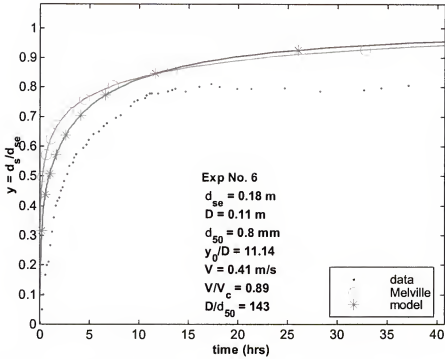


Figure B-6. UF Experiment 6: $V/V_c = 0.89$, $y_0/D = 11.14$, $D/d_{50} = 143$

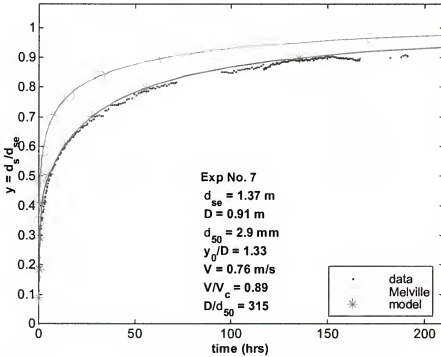


Figure B-7. UF Experiment 7: $V/V_c = 0.89$, $y_0/D = 1.33$, $D/d_{50} = 315$

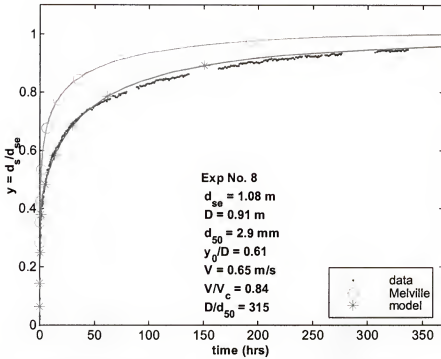


Figure B-8. UF Experiment 8: $V/V_c = 0.84$, $y_0/D = 0.61$, $D/d_{50} = 315$

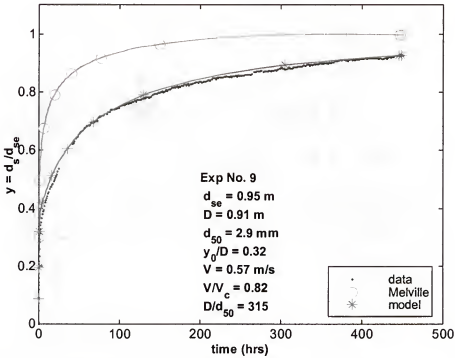


Figure B-9. UF Experiment 9: $V/V_c = 0.82$, $y_0/D = 0.32$, $D/d_{50} = 315$

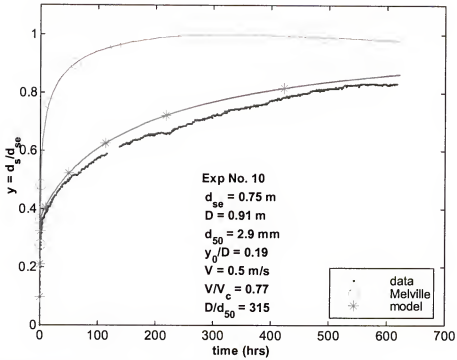


Figure B-10. UF Experiment 10: $V/V_c = 0.77$, $y_0/D = 0.19$, $D/d_{50} = 315$

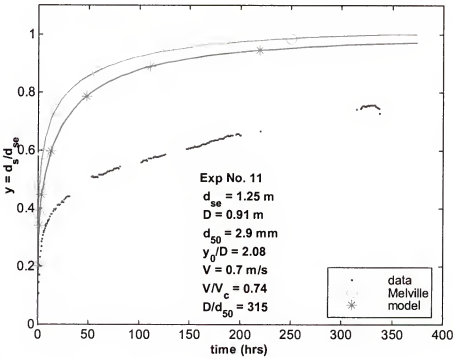


Figure B-11. UF Experiment 11: $V/V_c = 0.74$, $y_0/D = 2.08$, $D/d_{50} = 315$

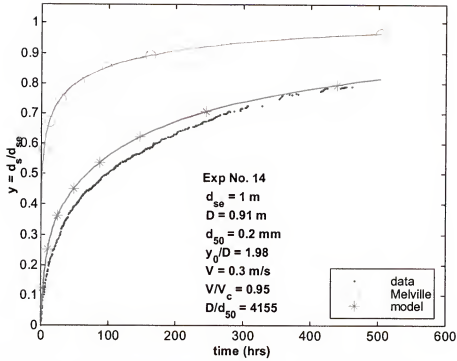


Figure B-12. UF Experiment 14: $V/V_c = 0.95$, $y_0/D = 1.98$, $D/d_{50} = 4155$

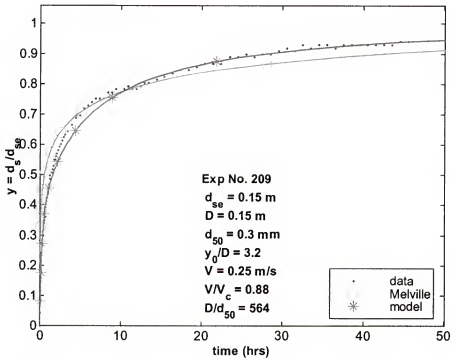
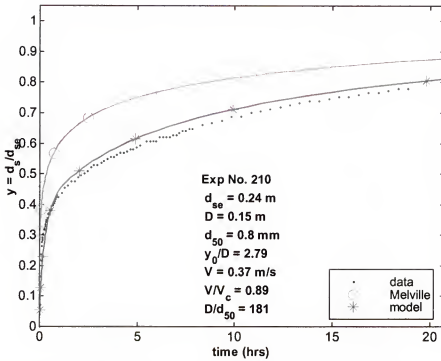
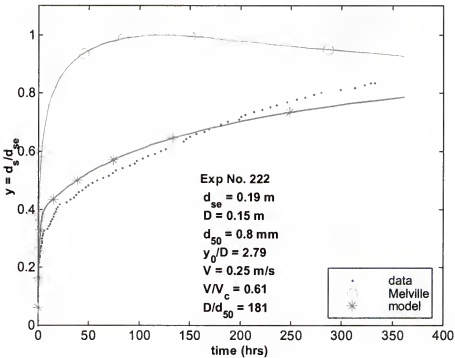
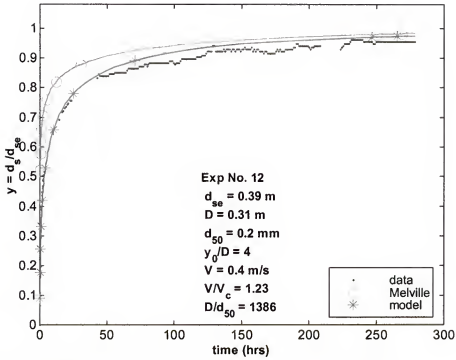
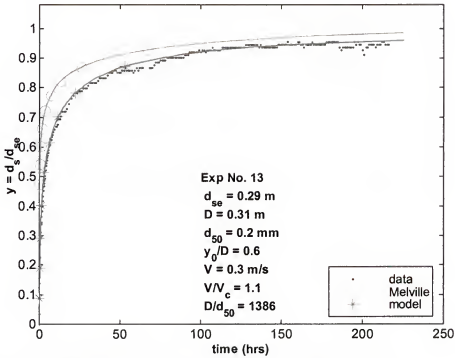


Figure B-13. UF Experiment 209: $V/V_c = 0.88$, $y_0/D = 3.2$, $D/d_{50} = 564$

Figure B-14. UF Experiment 210: $V/V_c = 0.89$, $y_0/D = 2.79$, $D/d_{50} = 181$ Figure B-15. UF Experiment 222: $V/V_c = 0.61$, $y_0/D = 2.79$, $D/d_{50} = 181$

B.2 Live Bed Scour Experiment Model Plots

Figure B-16. UF Experiment 12: $V/V_c = 1.23$, $y_0/D = 4$, $D/d_{50} = 1386$ Figure B-17. UF Experiment 13: $V/V_c = 1.1$, $y_0/D = 0.6$, $D/d_{50} = 1386$

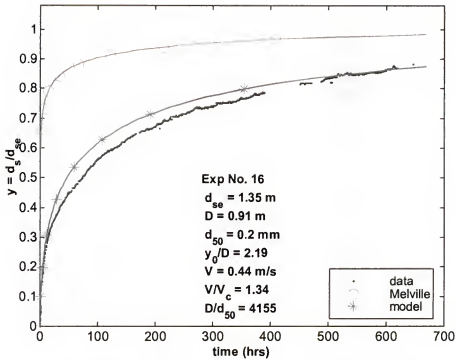


Figure B-18. UF Experiment 16: $V/V_c = 1.34$, $y_0/D = 2.19$, $D/d_{50} = 4155$

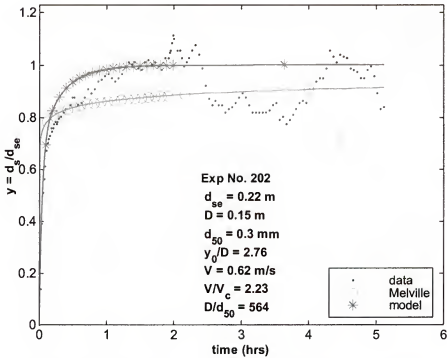
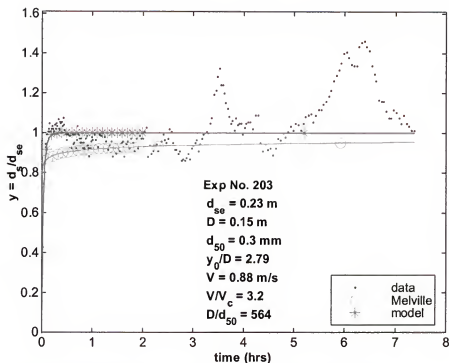
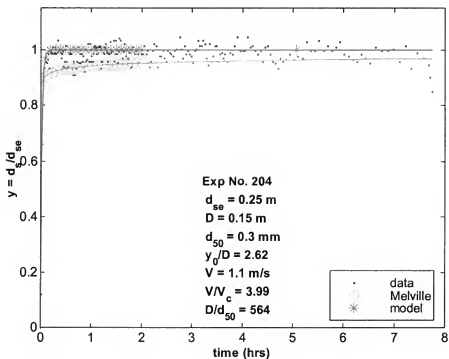
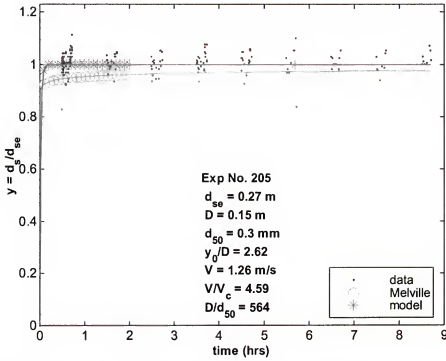
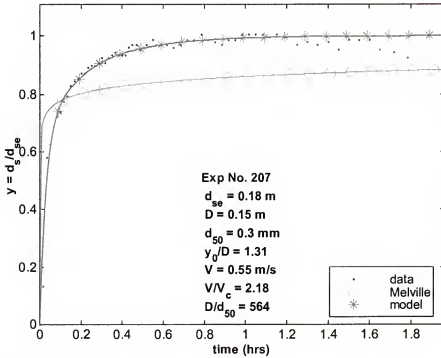


Figure B-19. UF Experiment 202: $V/V_c = 2.23$, $y_0/D = 2.76$, $D/d_{50} = 564$

Figure B-20. UF Experiment 203: $V/V_c = 3.2$, $y_0/D = 2.79$, $D/d_{50} = 564$ Figure B-21. UF Experiment 204: $V/V_c = 3.99$, $y_0/D = 2.62$, $D/d_{50} = 564$

Figure B-22. UF Experiment 205: $V/V_c = 4.59$, $y_0/D = 2.62$, $D/d_{50} = 564$ Figure B-23. UF Experiment 207: $V/V_c = 2.18$, $y_0/D = 1.31$, $D/d_{50} = 564$

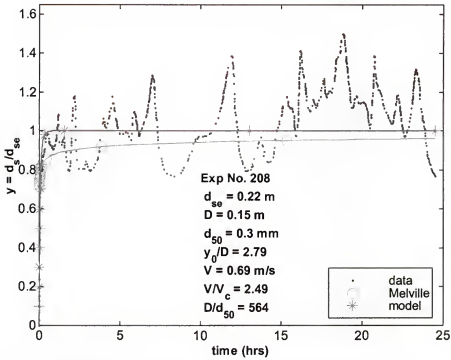


Figure B-24. UF Experiment 208: $V/V_c = 2.49$, $y_0/D = 2.79$, $D/d_{50} = 564$

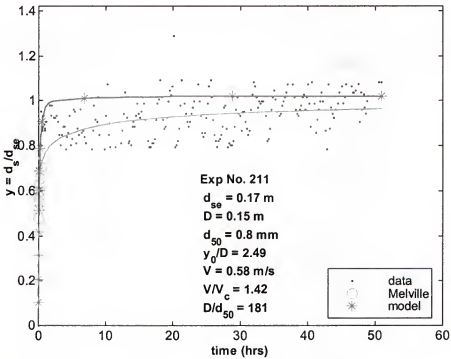
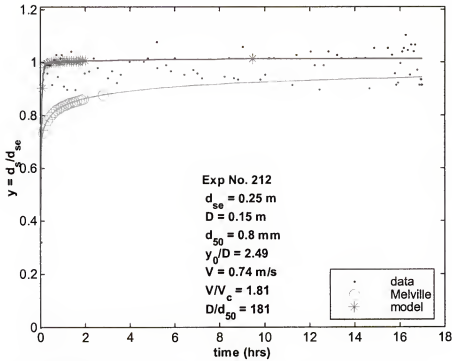
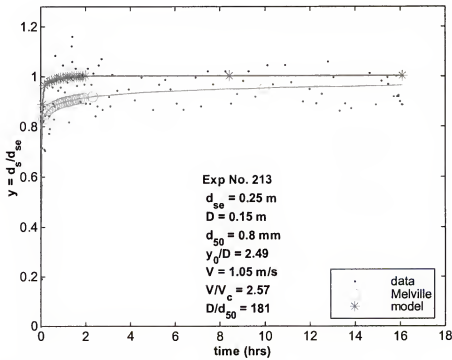
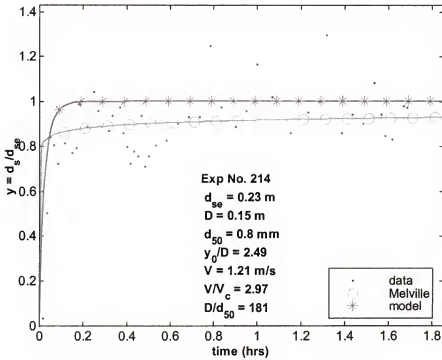
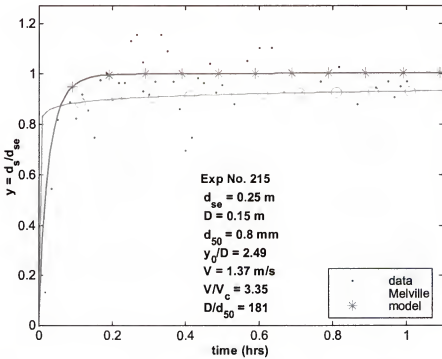


Figure B-25. UF Experiment 211: $V/V_c = 1.42$, $y_0/D = 2.49$, $D/d_{50} = 181$

Figure B-26. UF Experiment 212: $V/V_c = 1.81$, $y_0/D = 2.49$, $D/d_{50} = 181$ Figure B-27. UF Experiment 213: $V/V_c = 2.57$, $y_0/D = 2.49$, $D/d_{50} = 181$

Figure B-28. UF Experiment 214: $V/V_c = 2.97$, $y_0/D = 2.49$, $D/d_{50} = 181$ Figure B-29. UF Experiment 215: $V/V_c = 3.36$, $y_0/D = 2.49$, $D/d_{50} = 181$

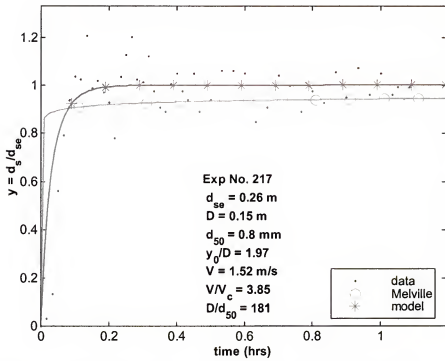


Figure B-30. UF Experiment 217: $V/V_c = 3.85$, $y_0/D = 1.97$, $D/d_{50} = 181$

APPENDIX C SPECIAL EQUATIONS AND CALCULATION METHODS

C.1 Water Density

Water density was determined from water temperature (T) and salinity (S) using the International Equation of State (1980) presented in Pond and Pickard (1983). For one standard atmosphere of pressure and temperature and salinity in degrees Celsius and parts per thousand respectively, this equation is

$$\begin{aligned} \rho = & 999.842594 + 0.0679395T - 0.00909529T^2 + 1.001685 \times 10^{-4}T^3 - \\ & 1.120083 \times 10^{-6}T^4 + 0.824493S - 0.0040899TS + 7.6438 \times 10^{-5}T^2S - \\ & 8.2467 \times 10^{-7}T^3S + 5.3875 \times 10^{-9}T^4S - 0.00572466S^{1.5} + 1.0227 \times 10^{-4}TS^{1.5} - \\ & 1.6546 \times 10^{-6}T^2S^{1.5} + 0.00048314S^2. \end{aligned} \quad (C-1)$$

C.2 Water Viscosity

Table C-1 lists the value of viscosity (μ) in fresh water (from Newman 1977) for various temperatures (C). For the data analysis and in the scour model, water viscosity was calculated by using a least squares fit equation for this data,

$$\mu = \left[\frac{1.789755 - 0.084T + 0.001524T^2}{2.9434 \times 10^{-5}T^3 + 3.091 \times 10^{-7}T^4} \right] \times 10^{-3} \text{ N-sec/m}^2. \quad (C-2)$$

Table C-1. Fresh water viscosity (Newman 1977)

T (C)	μ (N-s/m ²)
0	1.790×10^{-3}
5	1.520×10^{-3}
10	1.310×10^{-3}
15	1.139×10^{-3}
20	0.998×10^{-3}
25	0.887×10^{-3}
30	0.796×10^{-3}

Table C-2 lists the values of viscosity for varied salinities and temperatures.

Table C-2. Viscosity of seawater ($\times 10^{-3}$ N-s/m²). (Kennish 1989)

		Salinity (ppt)				
		5	10	20	30	40
Temperature (C)	0	1.803	1.817	1.844	1.887	1.883
	5	1.528	1.542	1.567	1.592	1.617
	10	1.319	1.331	1.403	1.380	1.403
	15	1.149	1.160	1.183	1.206	1.229
	20	1.015	1.026	1.047	1.070	1.092
	25	0.901	0.911	0.931	0.952	0.974
	30	0.811	0.822	0.840	0.860	0.877

Combining Tables C-1 and C-2 gives

$$\mu = \left[\begin{array}{l} 1.789755 - 0.084T + 0.001524T^2 - \\ 2.9434 \times 10^{-5} T^3 + 3.091 \times 10^{-7} T^4 + \\ 0.002412 S - 3.829 \times 10^{-6} S^2 \end{array} \right] \times 10^{-3} \text{ N-sec/m}^2. \quad (\text{C-3})$$

C.3 Logarithmic Velocity Profile

Conversion between friction velocities and depth averaged velocities required the assumption of a logarithmic velocity profile with depth (Sleath 1984).

The bed roughness (k_s) is given by $k_s = RRd_{50}$, where RR is the relative roughness and d_{50} is the median sediment diameter. The corresponding Reynolds Number is

$Re = u_* k_s / \nu$ where the friction or shear velocity (u_*) is given by $u_* = \sqrt{\tau_b / \rho}$ and τ_b is the bed shear stress. The basic formula for velocity as a function of depth, $u(z)$, in a fully developed flow with a logarithmic profile is

$$u(y) = \frac{1}{\kappa} u_* \ln \left(\frac{z}{z_0} \right) = 2.5 u_* \ln \left(\frac{z}{z_0} \right) \quad (\text{C-4})$$

where κ is von Karman's constant and $\kappa = 0.4$, z is the height above the bed and z_0 is the hypothetical zero velocity bed level. The depth averaged velocity (V) can be found by integrating Eq. C-4 over the depth (h) to give

$$V = 2.5u_* \ln\left(\frac{h}{2.72z_0}\right). \quad (C-5)$$

To find z_0 the flow is divided into regimes according to the bed Reynolds Number. For $Re < 5$ the flow is considered "smooth" and $z_0 = \nu/9u_*$. Eq. C-6 becomes

$$V = 2.5u_* \ln\left(\frac{3.31u_*h}{\nu}\right). \quad (C-7)$$

For $Re > 70$ the bed is considered "rough" and $z_0 = k_s/30$ to give

$$V = 2.5u_* \ln\left(\frac{11h}{k_s}\right). \quad (C-8)$$

For flows where $5 < Re < 70$, the flow is in transition and z_0 is a function of Re .

Fitting Figure 1.12 of Sleath (1984) gives the approximation

$$\frac{z_0}{k_s} = 1 \times 10^{-3} \left[-6 + 2.9Re - 0.59Re \ln(Re) + 0.002Re^2 + 0.11/Re \right]. \quad (C-9)$$

When the shear velocity is known the Reynolds number can be determined and the solution of Eq. C-5 and C-9 is straightforward. If the depth averaged velocity is known and the shear velocity must be determined, the equations must be solved by iterating between the u_* , the Reynolds Number, and the appropriate equation until all three match.

C.4 Shields Curve Approximation

Expanding the critical shear velocity (u_{*c}) gives

$$u_{*c} = \sqrt{\frac{\tau_c}{\rho}} = \sqrt{\frac{\tau_c}{\rho(sg-1)gd_{s0}}} (sg-1)gd_{s0} = \sqrt{\theta_c(sg-1)gd_{s0}} \quad \text{and}$$

$$\frac{u_*d_{s0}}{\nu} = \sqrt{\theta_c \frac{(sg-1)gd_{s0}^3}{\nu^2}}.$$

The non-dimensional sediment diameter (d_*) is defined as $d_* = d_{s0}[(sg-1)g/\nu^2]^{1/3}$ and

from Figure C-1

$$\theta_c = f\left(\frac{u_* d_{50}}{\nu}\right) = f\left(\theta_c \frac{(sg-1)gd_{50}^3}{\nu^2}\right) = f(\theta_c d_*^3).$$

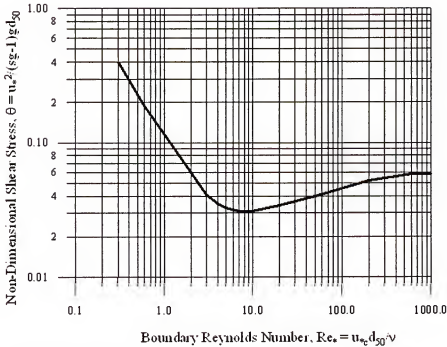


Figure C-1. Shields Diagram

Factoring out θ_c leaves $\theta_c = f(d_*)$ and the curve in Figure C-1 can be converted to a curve of θ_c versus d_* by calculating d_* from $d_* = \theta_c^{-1}(u_* d_{50}/\nu)^2$.

With this relationship, Van Rijn (1993) made the following approximation:

$$\begin{aligned} 1 \leq d_* \leq 4, & \quad \theta_c = 0.24d_*^{-1} \\ 4 < d_* \leq 10, & \quad \theta_c = 0.14d_*^{-0.64} \\ 10 < d_* \leq 20, & \quad \theta_c = 0.04d_*^{-0.1} \\ 20 < d_* \leq 150, & \quad \theta_c = 0.013d_*^{0.29} \\ d_* > 150, & \quad \theta_c = 0.055 \end{aligned}$$

Refitting the Shields curve data gives the following equations

$$\begin{aligned} 0.22 < d_* < 150, & \quad \theta_c = -0.005 + 0.0023d_* - 0.000378d_* \ln(d_*) + 0.23/d_*, \\ & \quad (C-10) \\ d_* > 150, & \quad \theta_c = 0.0575. \end{aligned}$$

Figure C-2 compares these two approximations with the data sets of various researchers, e.g. Gilbert (1914), Shields (1936), White (1940), Ho (1935), Meyer-Peter and Mueller (1948), Neill (1967). The advantage of Eq. C-10 is in the simplicity for programming and spreadsheet calculations.

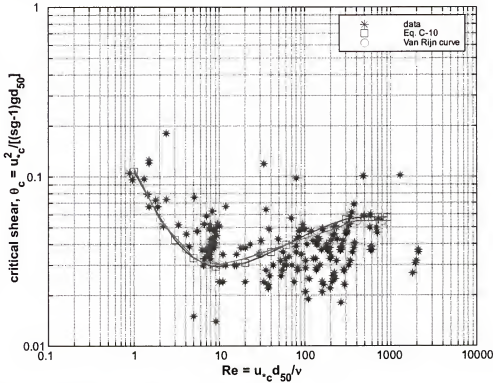


Figure C-2. Shields approximations comparison

C.5 Equilibrium Scour Depth

From Sheppard (2002a) the equilibrium scour depth can be calculated using three regions based on V/V_c . In the clear water region, $V/V_c < 1$ and the equilibrium scour depth equation is

$$\frac{d_{sc}}{D} = c_s f_1 \left(\frac{y_0}{D} \right) f_2 \left(\frac{V}{V_c} \right) f_3 \left(\frac{D}{d_{50}} \right). \quad (C-11)$$

The functions $f_1(y_0/D)$, $f_2(V/V_c)$ and $f_3(D/d_{50})$ are

$$f_1\left(\frac{y_0}{D}\right) = \tanh\left[c_1\left(\frac{y_0}{D}\right)^{c_2}\right], \quad f_2\left(\frac{V}{V_c}\right) = 1 - \left[\frac{\ln(V/V_c)}{\ln(V/V_c|_0)}\right]^2, \text{ and}$$

$$f_3\left(\frac{D}{d_{50}}\right) = \frac{c_3 + c_4}{c_3 \exp[c_4(x - x_{pk})] + c_4 \exp[-c_3(x - x_{pk})]}.$$

where $x = \log_{10}(D/d_{50})$ and $x_{pk} = \log_{10}(D/d_{50}|_{pk})$. In the V/V_c function, $V/V_c|_0$ is defined as the minimum value of V/V_c for which scour occurs. Sheppard (2002a) sets this value at 0.47. In the D/d_{50} function, x_{pk} is defined as the log of the value of D/d_{50} for which the function has a maximum. Sheppard sets this value at 44. The values for the other coefficients are $c_1 = 1.0$, $c_2 = 0.4$, $c_3 = 2.6$, $c_4 = 0.45$ and $c_5 = 2.5$.

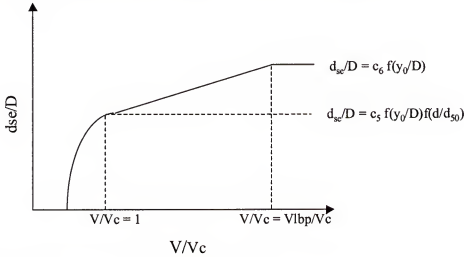


Figure C-3. Flow regions for equilibrium scour calculations

For live bed flows there are two regions in V/V_c . The first begins at $V/V_c = 1$ and ends when the velocity of the flow becomes high enough for the bed forms to plane out. The velocity at this point is known as the live bed peak velocity, V_{lbp} , and the flow intensity at this point is V_{lbp}/V_c . In the region for which $V/V_c > V_{lbp}/V_c$ the scour depth is only dependent on the pile diameter and, for shallow flows, the aspect ratio (y_0/D).

The live bed equation is (for flow intensities in the range $1 < V/V_c < V_{lbp}/V_c$)

$$\frac{d_{sc}}{D} = f_1\left(\frac{y_0}{D}\right) \left[c_6 \left(\frac{V - V_c}{V_{lbp} - V_c} \right) + c_5 f_3 \left(\frac{D}{d_{50}} \left(\frac{V_{lbp} - V}{V_{lbp} - V_c} \right) \right) \right]$$

where $f_1(y_0/D)$ and $f_3(D/d_{50})$ are from the clear water equation and $c_6 = 2.2$. For

$$V/V_c > V_{lbp}/V_c$$

$$\frac{d_{sc}}{D} = c_6 f_1\left(\frac{y_0}{D}\right).$$

To determine the value of V_{lbp} two methods are employed and the higher value is used. From Van Rijn (1993), the flat bed condition occurs when $T = (\tau_b - \tau_c)/\tau_c \geq 25$ and $V/\sqrt{gh} \geq 0.8$; where τ_b is the bed shear stress, τ_c is the critical bed shear stress, V is the depth averaged velocity, and h is the water depth.

To determine the bed shear stress, Van Rijn uses the Chezy Equation

$$\tau_b = \rho g (V/C_z)^2, \text{ where } C_z = 5.75 \sqrt{g} \log_{10}(12h/k_s). \text{ Therefore, } V_{lbp} \text{ is the depth}$$

averaged velocity that meets both of the following criteria:

$$V \geq 0.8\sqrt{gh} \text{ and } V \geq 5.75 \log_{10}\left(\frac{12h}{k_s}\right) \sqrt{\frac{26\tau_c}{\rho}}.$$

Snamenskaya (1969) plotted Froude Number ($V/\sqrt{gh}$) against the ratio of depth averaged velocity to sediment fall velocity (V/w) and mapped regions where various bed forms occur (Figure C-4). Fitting a curve through the flat bed region (region 6) gives

$$\text{for } 4 \leq V/w \leq 500, Fr = 0.8251 - 0.00285[\ln(V/w)]^2 \text{ and}$$

$$\text{for } 500 \leq V/w \leq 1500, Fr = 0.6968 - 2.2752(1 + 0.261662 V/w)^{-1}.$$

To solve for the live bed peak velocity by this method, a velocity (V) is assumed.

The Froude Number is then calculated and compared to the Froude Number based on Snamenskaya's curve. The velocity is then iterated until the Froude Numbers match.

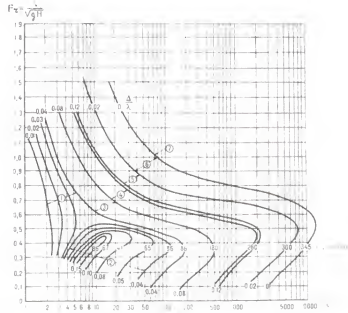


Figure C-4. Froude Number vs. V/w with bed form regions (Snamenskaya 1969)

C.6 The Runge-Kutta Calculation Scheme

To solve the model equation numerically, an adaptive step sized fifth order Runge-Kutta routine was used and compared to an embedded fourth order equation. If the error was not within a preset tolerance, the step size was halved and the solution was recalculated. This process was repeated until the solutions agreed.

The method was obtained from Press et al. (1986) and used the Runge-Kutta-Fehlberg method with Cash-Karp parameters (Table C-3). For the differential equation

$$dy/dt = f(x, y), \text{ the general form of the fifth order Runge-Kutta is } y_{n+1} = y_n + \sum_{i=1}^6 c_i k_i$$

where

$$k_1 = hf(x_n, y_n),$$

$$k_2 = hf(x_n + a_2 h, y_n + b_{21} k_1),$$

$$k_3 = hf(x_n + a_3 h, y_n + b_{31} k_1 + b_{32} k_2),$$

$$k_4 = hf(x_n + a_4 h, y_n + b_{41} k_1 + \dots + b_{43} k_3),$$

$$k_5 = hf(x_n + a_5 h, y_n + b_{51} k_1 + \dots + b_{53} k_3), \text{ and}$$

$$k_6 = hf(x_n + a_6 h, y_n + b_{61} k_1 + \dots + b_{63} k_3).$$

The embedded fourth-order equation is $y_{n+1}^* = y_n + \sum_{i=1}^6 c_i^* k_i$. The error can be calculated

from $\Delta = y_{n+1} - y_{n+1}^* = \sum_{i=1}^6 (c_i - c_i^*) k_i$.

Table C-3. Cash-Karp parameters for embedded Runge-Kutta Method

i	a_i		b_{ij}				c_i	c_i^*
1	0	0	0	0	0	0	<u>37</u>	<u>2825</u>
							378	27648
2	0.2	0.2	0	0	0	0	0	0
3	0.3	0.075	0.225	0	0	0	<u>250</u>	<u>18575</u>
							621	48384
4	0.6	0.3	-0.9	1.2	0	0	<u>125</u>	<u>13525</u>
							594	55296
5	1	<u>-11</u>	2.5	<u>-70</u>	<u>35</u>	0	0	<u>277</u>
		54		27	27			14336
6	0.875	<u>1631</u>	<u>175</u>	<u>575</u>	<u>44275</u>	<u>253</u>	<u>512</u>	0.25
		55296	512	13824	110592	4096	1771	
j =		1	2	3	4	5		

C.7 Scour Model Input and Calculation Requirements

The minimum inputs to the scour model are depth averaged velocity (V), flow depth (y_0), cylinder diameter (D), median sediment size (d_{50}), density (ρ_s), porosity (p) and submerged angle of repose (ϕ) and water density (ρ) and kinematic viscosity (ν). Assumed input parameters are bed relative roughness, the erosion zone width divided by the cylinder diameter (n) and the half-angle of the perimeter of the erosion zone (β_e) and the minimum flow intensity for which scouring will occur (ϵ).

Given these input parameters and the assumptions listed in Chapter 6, the calculations required for solving the scour model of Eq. 4-27 are listed in Table C-4. Using the parameters calculated in Table C-4, at each $y_n = d_{s,n}/d_{se}$ the effective shear stress is calculated using shear stress model of Table 4-2 and dy_n/dt is calculated using Eq. 4-27. Then, the solution for $y(t)$ is incremented in accordance with the numerical solution scheme (section C.6).

Table C-4. Model input and calculation summary

Inputs: $V, y_0, D, d_{50}, p, \phi, \rho_s, \rho, \nu$

Assumed: $RR, n, \beta_e, \varepsilon$

Calculated parameters:

$$sg = \rho_s/\rho, k_s = RRd_{50}, d_* = f(d_{50}, sg, \nu)$$

$$\theta_c = f(d_*) \text{ (section C.4), } u_{*c} = \sqrt{\theta_c (sg - 1) g d_{50}}$$

$$V_c = f(u_{*c}, d_{50}, k_s, y_0, \nu) \text{ (section C.3)}$$

$$V_{ibp} = f(V, w, y_0, k_s, d_{50}, \theta_c, \rho, sg) \text{ (section C.5)}$$

$$d_{sc} = f(V/V_c, V/V_{ibp}, y_0/D, D/d_{50}) \text{ (section C.5)}$$

$$u_* = f(V, d_{50}, k_s, y_0, \nu) \text{ (section C.3), } \theta_u = u_*^2 / [(sg - 1) g d_{50}]$$

$$\theta_0' = \theta_u / \varepsilon^2 \text{ (} \varepsilon \sim 0.47, \text{ Sheppard 2002a)}$$

$$q_{in} = f(\theta_u, \theta_c, d_{50}, sg)^a$$

Transport and Shear Model Calculations ^b

$$C = f(\theta_0', \theta_c, D/d_{50}, d_{50}, sg)$$

$$y_p = f(V/V_c, D/d_{50}, y_0/D)$$

$$\theta_p' = f(V/V_c, D/d_{50}, y_0/D, \theta_0')$$

$$y_b = f(V/V_c, D/d_{50}, y_0/D, V, y_0, sg)$$

$$\theta_b' = f(V/V_c, D/d_{50}, y_0/D, V, y_0, sg, d_{50}, \theta_c)$$

^a see Table 4-1

^b see Table 4-3

APPENDIX D DESCRIPTION OF EXPERIMENTS AND INSTRUMENTATION

D.1 Clear Water Scour Experiments Instrumentation

In the tests at the Conte USGS-BRD Laboratory the flow discharge and depth-averaged velocity were controlled with a weir located at the downstream end of the flume. The height of the weir was set for the desired water depth and flow discharge for each experiment.

Flow velocities were directly measured at two locations with Marsh-McBirney electromagnetic flow meters: a model 523 with a 13 mm diameter sensor and a model 511 with a 38 mm diameter sensor. The sensors were placed 5 m upstream from the center of the test structure and 1.5 m to the side of the center of the test structure. The vertical position of the meters was set at 40% of the water depth from the bed since the velocity at this depth is approximately equal to the depth-averaged velocity for a fully developed logarithmic velocity profile.

Velocities at the same depth as the Marsh-McBirney sensors were also measured using an impeller type current meter (Ott-meter). The Ott meter was a C 2 Small Current Meter and the propeller was a No.1-113040 model with a Z-30 quartz counter. This instrument was used periodically during the experiments to check the electromagnetic meters. The accuracy of the Marsh-McBirney meters was 2% of the reading as reported by the manufacturer.

A Bern-Hydrawater water level instrument, which uses a near bottom mounted pressure transducer, measured water level for the experiments. The water level was measured at the same location for all of the experiments downstream of the test structure and approximately 7 m upstream of the weir. The accuracy of the water level meter was 0.01% of the full-scale readings according to the manufacturer.

An AC2626K Analog Devices temperature sensor was used to measure water temperature. The temperature was measured at the same location for all experiments, approximately 3 m from the test structures. The accuracy of the temperature measurement was estimated to be ± 1 C.

The scour hole depth was monitored by video cameras inside (and on some occasions outside) the cylinders and by arrays of acoustic transponders attached to the cylinders just below the water surface. Two miniature video cameras were mounted on a platform inside the cylinders. The cameras were moved vertically as the experiment progressed to follow the level of the water-sediment interface at the base of the cylinder. Length scales were attached to the inside of the cylinders in view of the cameras so that quantitative scour depth measurements could be obtained from the video images. The video signal from the cameras was sent to a VHS recorder and a control system was used to set the duration of the recordings and the intervals between recordings. A switching circuit switched the recorder input between the two cameras. Down facing floodlights near the cylinder were also turned on and off by the controller in phase with the recordings. Due to its size, the large 0.915 m diameter cylinder was flooded with water during the experiments and the cameras for this cylinder were mounted in a waterproof housing. The 0.114 m and 0.305 m diameter cylinders were water-tight.

In addition to the video, scour depth was measured by three arrays of acoustic transponders attached to the cylinder just below the water surface. This system provided scour hole depth measurements at 12 locations along three radial lines throughout the experiments. Because of cylinder size differences, two different MTA systems were used; one for the 0.114 m and 0.305m cylinders and one for the 0.915 m cylinder.

Each system consisted of three arrays and each array contained four crystals. Each crystal produced a narrow acoustic beam (2.5 cm in diameter at the transducer with a spread angle of approximately 1.5 degrees) that was used to measure distance from the transducer to the bed. The transducers are called Multiple Transducer Arrays or simply MTAs (Jette and Hanes, 1997). They were positioned at the front of the structure and at angles of 83° from the front as shown in Figure D-3. Aluminum bands were used to attach the arrays to the cylinders. Their distance from the bed varied depending on the water (and scour hole) depth, but care was taken to locate them as far as possible from the bed so as not to interfere with the scour processes.

A bridge across the flume provided horizontal support for the tops of the test structures and a platform for the computers and data acquisition systems.

D.2 Live Bed Scour Experiments Instrumentation

Prior to the experiments, tests were conducted to obtain the relationship between pump speeds and flow discharge. Velocity profiles with respect to depth were made using an acoustic Doppler velocimeter. These profiles were then integrated to obtain flow discharge for a range of pump speeds.

During the tests, water depth was measured at the test section with scales on the glass walls of the flume. A sectionally averaged velocity was estimated from this depth

and the flow discharge (determined from the pump speed). The velocity at the test section was regulated by adjusting the pump speed.

Water temperature was periodically measured by a standard glass thermometer and recorded.

As in the clear water scour tests, the scour depths were measured by both video cameras located inside the test cylinder and an array of three acoustic transponders (MTAs) attached to the outside of the cylinder. This arrangement and the MTAs were identical to that used in the clear water scour tests. Figures D-4 and D-6 show the transponder configuration. Figures D-5 and D-7 show the configuration internal cameras and Figure D-5 shows the orientation of the internal cameras to the water flow in the flume.

Bed forms were a concern for the live bed tests, so an array of 22 acoustic transponders was used to measure the bed forms that propagated down the flume. Additionally, video cameras were mounted outside the flume (see Figure D-5) to monitor the bed and water elevations at the test section and to record the propagation of bed forms through the test area.

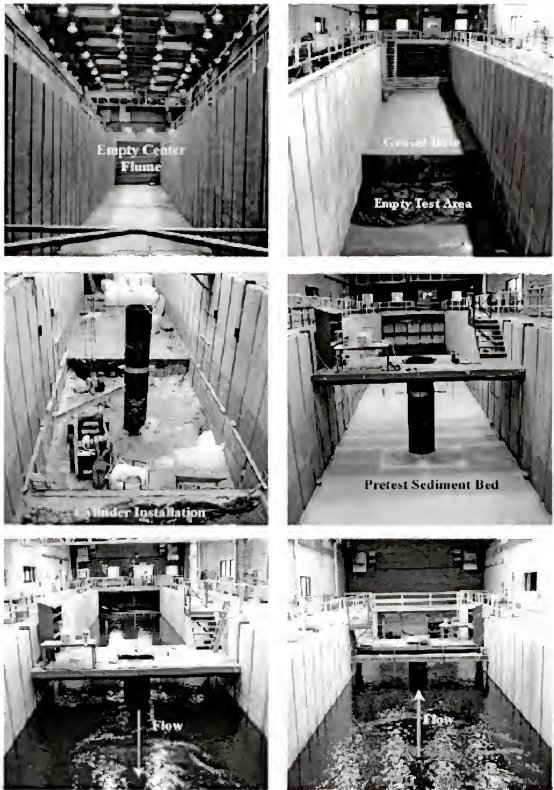


Figure D-1. Photographs of the Conte USGS-BRD Laboratory flume in operation

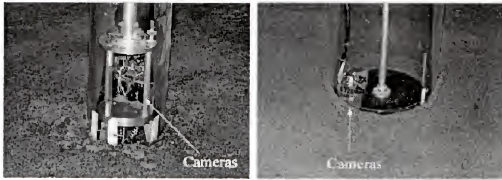


Figure D-2. Internal cameras mounted inside of 0.1 m (left) and 0.3 m (right) test cylinders



Figure D-3. Test cylinders with MTAs mounted: 0.1 m cylinder (left) and 0.9 m cylinder (right)

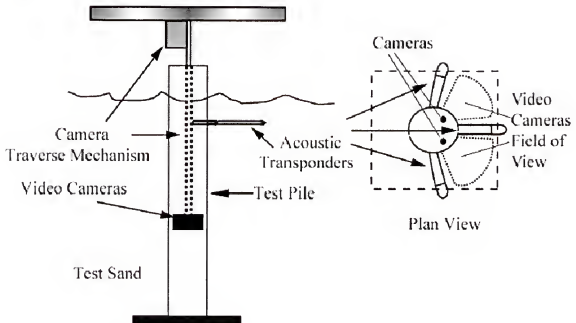
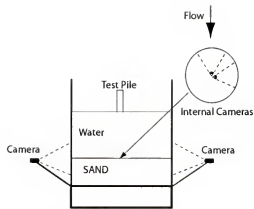


Figure D-4. Schematic of a test cylinder with mounted MTAs and cameras



Flume Cross-Section

Figure D-5. Cross-sectional schematic of the University of Auckland Hydraulics Laboratory Sediment Recirculating Flume with test cylinder and video cameras

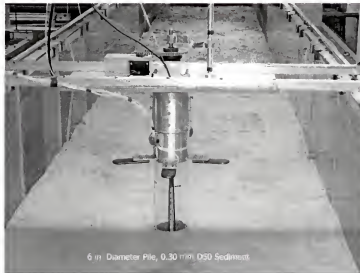


Figure D-6. Instrumented test cylinder installed in the University of Auckland flume prior to testing

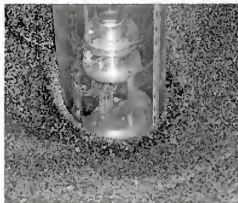


Figure D-7. Internal cameras and housing for the 0.152 m diameter test cylinder

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BIOGRAPHICAL SKETCH

William Miller Jr. was born in Newark, New Jersey in 1960. He received a Bachelor of Science in naval architecture and marine engineering from the University of Michigan in 1982, graduating cum laude. In 1997 he received a Master of Science degree in coastal and oceanographic engineering from the University of Florida, completing his thesis on the dynamic response of offshore structures.

He is a 20-year veteran of the United States Navy and Naval Reserve and currently holds the rank of Commander. As a Naval Officer he qualified in and served on submarines for ten years, completing tours as Sonar Officer, Main Propulsion Assistant and Weapons Department Head. He qualified a Nuclear Engineer and is a graduate of the U.S. Navy's Submarine School and graduate and former instructor of Navy's Nuclear Power Training Program.

He is authorized to wear three awards of the Navy and Marine Corps Commendation Medal and five awards of the Navy and Marine Corps Achievement Medal. These include awards Commander Miller received for his efforts in operations in the Middle East in response to the USS Cole attack in October 2000 and the terror attacks of September 2001.

William Miller Jr. is recently engaged to be married to Amy Hill and looks forward life as a husband and father.

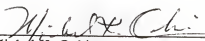
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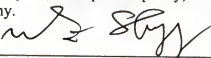
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